THESIS

Evaluation of Simplified Seismic Vulnerability Assessment Procedures for Reinforced Concrete Buildings

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Abstract

The need for a simple but rigorous seismic assessment procedure to predict damage to reinforced concrete buildings during a seismic event has been highlighted following the Canterbury Earthquake sequence. Such simplified assessment procedure, applied to individual structure or large building inventory, should not only have low requirement in terms of input information and involve straightforward analyses, but also should be capable to provide reliable predictive results within short timeframe.

This research provides a general overview and critical comparison of alternative simplified assessment procedures adopted in NZSEE 2006 Guidelines (Assessment and Improvement of the Structural Performance of Buildings in Earthquakes), ASCE 41-13 (Seismic Evaluation and Retrofit of Existing Buildings), and EN: 1998-3: 2005 (Assessment and Retrofitting of Buildings). Particular focus is given to the evaluation of the capability of Simplified Lateral Mechanism Analysis (SLaMa), which is an analytical pushover method adopted in NZSEE 2006 Guidelines. The predictive results from SLaMa are compared to damages observed for a set of reinforced concrete buildings in Christchurch, as well as the results from more detailed assessment procedure based on numerical modelling.

This research also suggests improvements to SLaMa, together with validation of the improvements, to include assessment of local mechanism by strength hierarchy evaluation, as well as to develop assessment of global mechanism including post-yield mechanism sequence based on local mechanism.

Abbreviations/Glossary

RC = reinforced concrete

C = concrete

DIR = direction

LV = Level

GLV = ground level

TLV = top level

PC = precast

IS = in-situ

D = deformed bars

R = plain bars

H = high strength bars

FR = frame

JO = joint

BE = beam

CO = column

WA = wall

SL = slab

FL = floor

DI = diaphragm

FO = foundation

SP = splice

ST = stirrups

TI = ties

DB = dowel bar

Acknowledgement

Author would like to especially acknowledge the great support and guidance provided by Professor Stefano Pampanin throughout thesis research duration. Author would also like to acknowledge the great support from Dr Ali Sahin Tasligedik (EQC), Dr Weng Yuen Kam (BECA), Dr Alejandro Amaris (CCC) and UC SAFER research group. Author has inspired from the research work carried out by SAFER research group members. The contributions of group members are shown in the following table (Table 0- 1). Future researches and investigations in the assessment field will be carried out by the group.

Table 0- 1: Acknowledgement to the contributions of SAFER research group

SAFER Research Group	Contribution of Each Member
Alberto Cuevas Ramiez	Research of material and component residual strength
Alessandro DePra	Study of Italian and European assessment code provisions
Amir Malek	Research of concrete properties and strengths
Arsalan Niroomandi	Research of column properties and strengths
Farhad Dashti	Research of shear wall properties and strengths
Giuseppe Loporcaro	Research of reinforcing steel properties and strengths
Manya Georgieva Deyanova	Research of column properties and strengths
Simona Bianchi	Study of Italian and European assessment code provisions

CHAPTER 1 Introduction

Canterbury (Christchurch) region was severely struck by a series of earthquakes, 4 September 2010 Mw 7.1 Darfield (Canterbury), 22 February 2011 Mw 6.2 Christchurch (Lyttelton) earthquake, 22 June 2011 Mw 6.0 Christchurch aftershock, and many lower-magnitude aftershocks. In the most catastrophic 22 February event, a large number of reinforced concrete buildings in the Christchurch Central Business District (i.e. CBD, defined by the four major avenues, Bealey, Fitzgerald, Moorhouse and Deans) were severely damaged, leading to 182 fatalities, 135 of which were the most unfortunate consequences of the complete collapse of two mid-rise reinforced concrete buildings.

Following the 22 February earthquake, field damage reconnaissance was immediately carried out by a structural research group from University of Canterbury, directly contributing to the emergency response and recovery activities under the purview of Civil Defence (CD), Ministry of Civil Defence and Emergency Management (MCDEM), Christchurch City Council (CCC). The reconnaissance was carried out in conjunction with Christchurch City Council (CCC)'s Building Safety Evaluation (BSE) process, following the NZSEE guidelines, and were only limited to multi-storey reinforced concrete buildings. The information obtained from the reconnaissance was recorded in CHCH CBD Database (or the further Refined Database, discussed in Chapter 6), providing an important source of information for academic researches.

Given the magnificent social and economic impacts caused by the earthquakes, urgent actions are required in aiming to remedy structural deficiencies that have been confirmed or newly discovered following the sequence of the earthquakes since 4 September 2010. For the pre70s (including 1970s) reinforced concrete buildings, one of the critical deficiencies confirmed is that these buildings lacked capacity design principles at local level (i.e. subassembly level, inadequate hierarchy of strength) and at global level (i.e. weak-column-strong-beam mechanism or soft-storey prone), as shown in Figure 1-1. Corresponding to this critical deficiency of the pre70s structures, severe joint damages and column sidesway mechanisms were observed after the seismic events. For the post 70s reinforced concrete buildings, as computer programmes for structural analyses have been significantly developed and become much greater use, the buildings of this period were designed and constructed with much more complex configurations and load paths. Hence, structural deficiencies due to plan or vertical irregularity, insufficient load paths, lack of redundancy, constraint of displacement (i.e. drift or ductility) capacity can be anticipated, as shown in Figure 1- 2. The damages observed were found to be consistent with these deficiencies. For the most of the damaged buildings from all periods, it has been found that the inadequacy in structural reinforcing detailing exists, for instances, lack of stirrups in joint regions or in columns, inadequate confinement in boundary regions of columns and walls, insufficient detailing for lap splices and anchorages, use of plain round bars pre 1970s, etc. Other structural deficiencies, lack of displacement compatibility considerations (e.g. beam-elongation and tearing-unseating of the flooring units, inefficient diaphragm action, potential collapse of interior gravity columns at low drift level, etc.), complex and dangerous interaction with heavy non-structural elements, weak foundation, inadequate material properties, have also been identified during the building reconnaissance. The critical structural deficiencies are summarised in Table 1- 1, and more details illustrated by structural drawings and photos of observed damages are presented in Appendix A10. More details associated with the observed damages are shown in Section 6.2.

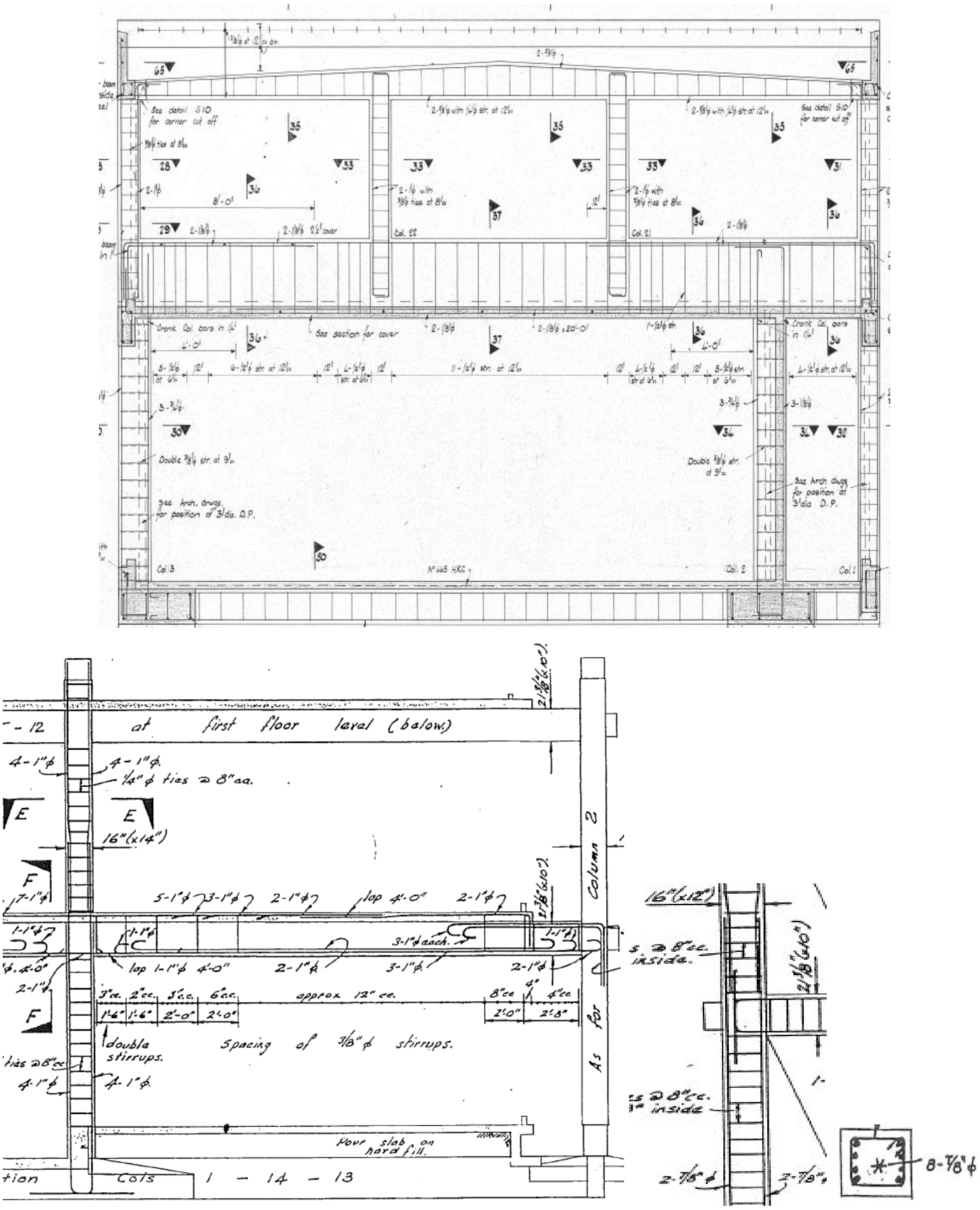
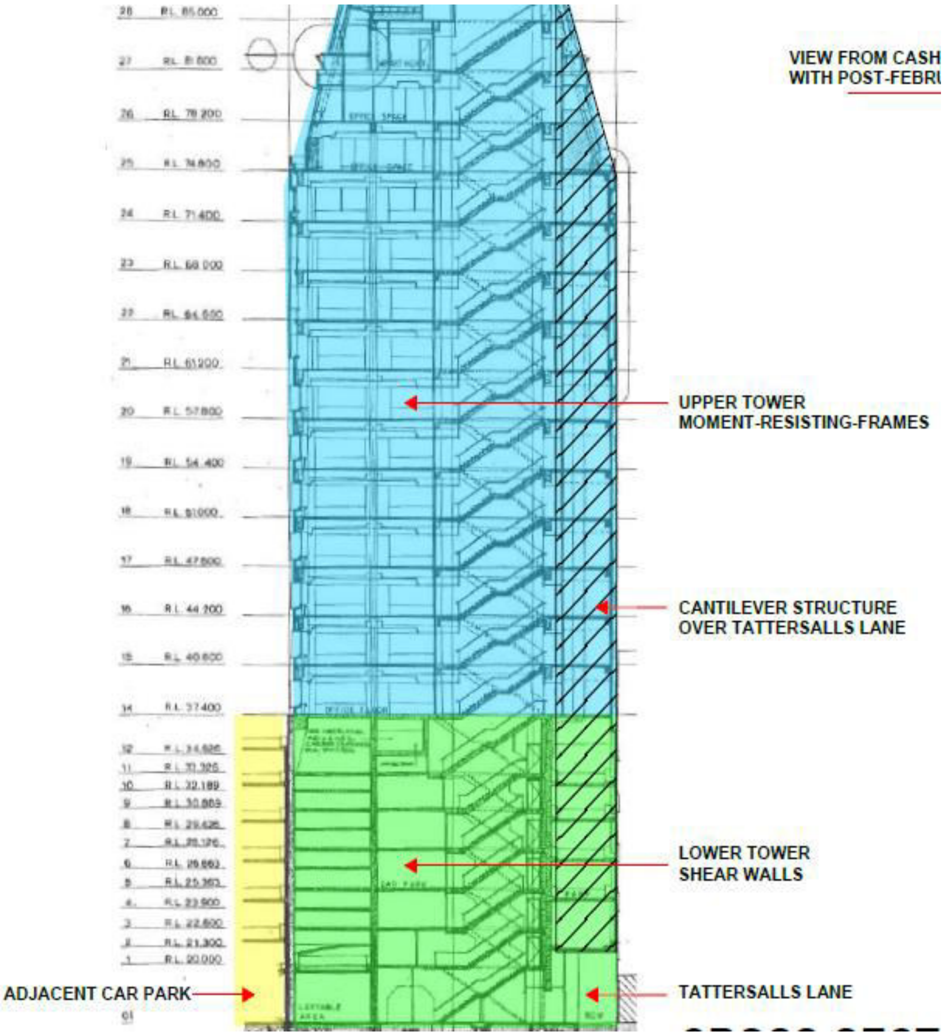
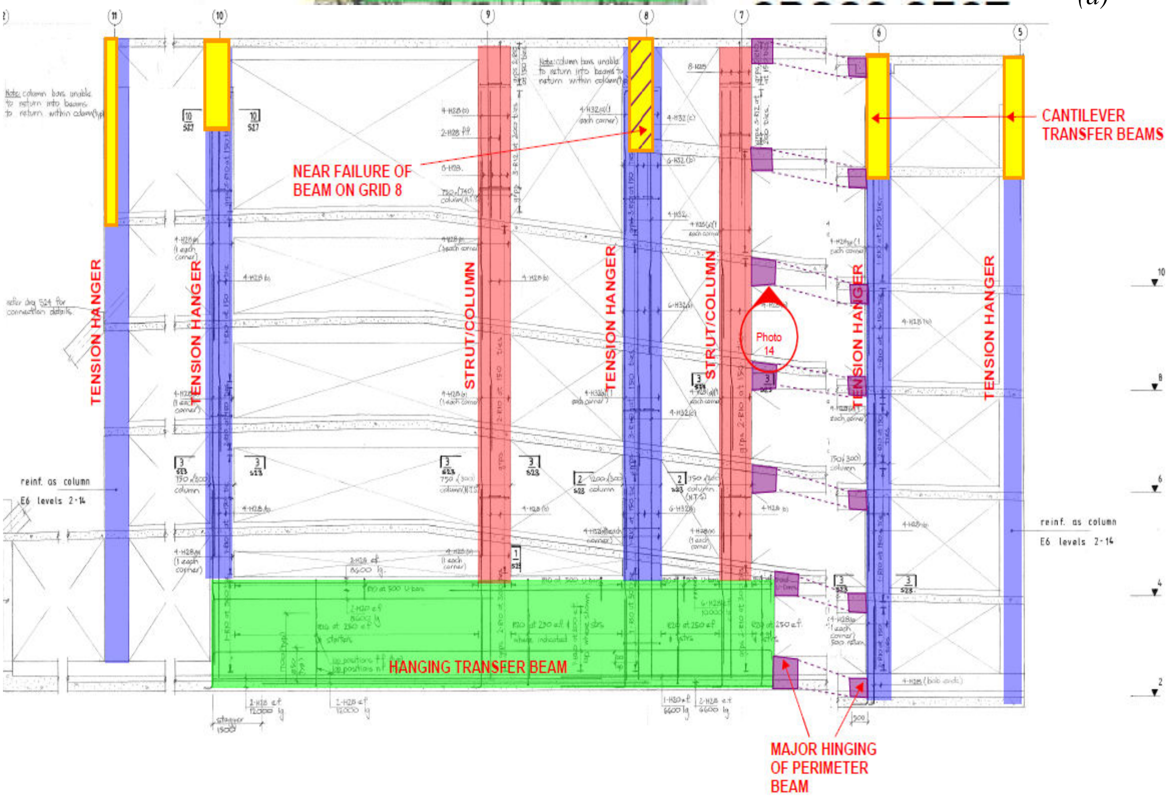


Figure 1- 1: Structural drawings of two pre 1970s buildings showing "weak-column-strong-beam" and inadequate reinforcing detailing of structural components (e.g. beams and columns)


VIEW FROM CASH
WITH POST-FEBRU
UPPER TOWER
MOMENT-RESISTING-FRAMES
CANTILEVER STRUCTURE
OVER TATTERSALLS LANE
LOWER TOWER
SHEAR WALLS
ADJACENT CAR PARK
TATTERSALLS LANE

(a)


CANTILEVER
TRANSFER BEAMS
NEAR FAILURE OF
BEAM ON GRID 8
TENSION HANGER
STRUT/COLUMN
Photo
14
HANGING TRANSFER BEAM
MAJOR HINGING
OF PERIMETER
BEAM

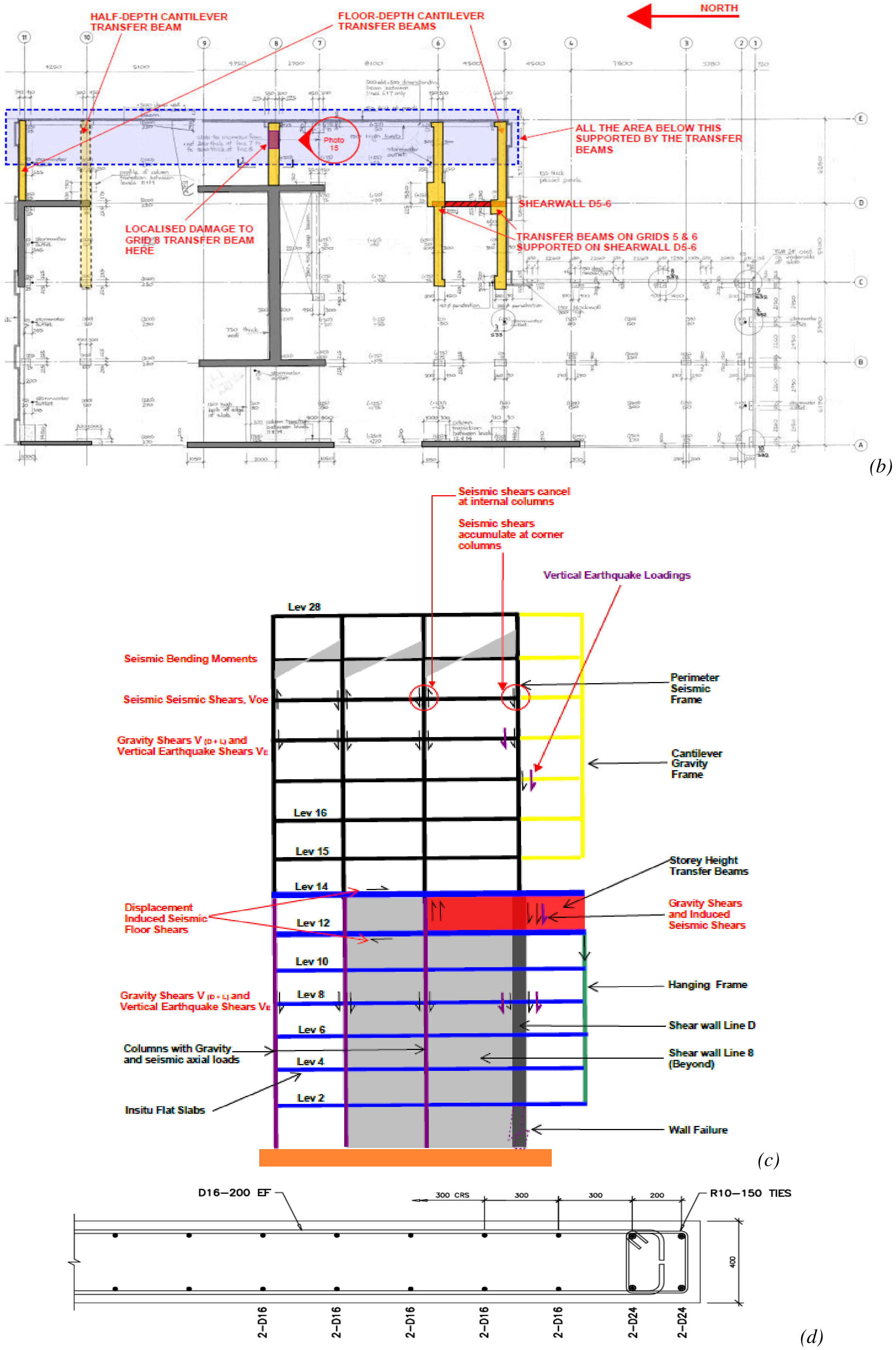


Figure 1- 2: Study of a 1980s building showing deficiencies due to (a) irregularity in elevation; (b) irregularity in plan; (c) insufficient load path; (d) lack of reinforcing detailing of structural components (e.g. structural wall)

Table 1- 1: Summary of structural deficiencies and observed damages (see Section 6.2 and Appendix A10)

Year of Built	Components or Global Structure	Structural Deficiencies	Observed Damages
Pre70s (incl.70s)	**Beams**	Poor confinement and shear reinforcement in beams	Flexural or shear type of damages to beams
		Inadequate anchorage	(Shown in Joint part)
		Inadequate splice detailing	Beam failure at splice
		Use of plain round bars	Slip of reinforcing bars, failure of beams
Pre70s (incl.70s)	**Joints**	Absence of horizontal and/or vertical transverse reinforcement	Damage to joint area
		Inadequate anchorage of beam longitudinal bars into the joint	Damage to joint area
		Lack of reliable joint shear transfer mechanism beyond diagonal cracking - use of plain round bars	Slip of reinforcing bars, failure of joints
Pre70s (incl.70s)	**Columns**	Inadequate confinement at the plastic hinge. Not all of the bars of the longitudinal reinforcement are confined with stirrups.	Shear failure of the column at the plastic hinge Buckling of the longitudinal reinforcement at the plastic hinge
		Inadequate shear reinforcement	Failure of the columns due to significant decrease in the flexural capacity of the plastic hinge
		Column shear span shortening due to masonry infills Column shear span shortening due to stiff facade non-structural elements	Shear failure of the columns due to short-column phenomenon
		Relocation of the plastic hinge in columns due to stiff non-structural elements	Flexural cracks along a length greater than the expected plastic hinge length
Pre70s (incl.70s)	**Walls**	Inadequate longitudinal reinforcement, i.e. single-layer	Buckling of wall
		Poor confinement and shear reinforcement in walls	Wall boundary zone compression crushing and buckling failure
		Inadequate lap splice detailing	Buckling failure
		Excessive wall slenderness ratio (wall height-to-thickness ratio	Buckling failure
Pre70s (incl.70s)	**Global Structure**	Lack of capacity design: weak-column-strong-beam mechanism, soft-storey prone	Severe damages to columns or joints, and soft-storey mechanism
Post80s (incl.70s)	**Beams**	Beam elongation and precast floor diaphragm failure	Failure of diaphragm due to loss of seating support
Post80s (incl.70s)	**Columns**	Lap-splicing with not enough length and confinement. More often away from the plastic hinge region	Damages due to the compromised continuity of the element
		Not enough confinement at the plastic hinge region of columns with high axial load ratio	Shear-axial failure of columns
		Not enough transverse reinforcement in circular columns to resist torsion	Torsional cracks
Post80s (incl.70s)	**Walls**	Irregular shapes	Out-of-plane damages
		Inadequate confinement	Crushing, spalling of concrete; bar buckling; out-of-plane failure
Post80s (incl.70s)	**Global Structure**	Plan irregularity	Damages due to torsional effect to components
		Vertical irregularity	Column sidesway mechanism

Given the target of remedying the structural deficiencies, apart from that New Zealand seismic design provisions need to be improved, seismic assessment guidelines also need to be reviewed and improved. A robust assessment procedure should be able to achieve the objectives shown as following, at both pre-earthquake and post earthquake stages:

Pre-earthquake Stage: (Vulnerability Examination and Remediation)

- To predict how prone the structure to damage under different seismic levels
- To help to determine strengthening solutions for the structure

Post-earthquake Stage: (Damage Investigation and Retrofit/Rebuilt)

- To investigate to what level the structure was damaged during the seismic event
- To help to determine retrofitting schemes applied to the structure
- To provide vital information for loss and risk estimation

It has been acknowledged that in the assessment of a large building inventory (e.g. the CHCH RC Building Database, or the Refined Building Database), the application of comprehensive assessment procedures may require considerable amounts of time and research efforts to collect and compile data, funds to back up the research and study, and powerful computing or analysing tools. Under some circumstances where the available data, time, money or computing tools are limited to some extent, comprehensive assessment procedures may become impractical and unjustified. Therefore, simplified procedures retaining the virtues of the comprehensive procedures are preferred. Compared to comprehensive procedures, simplified procedures indeed show great advantages in less requirement of input information, great saving in time and money, and may adopt much simpler analysis approaches. However, the problem of feasibility and efficiency of such procedures arises; thereby, the focus of this research – evaluation of simplified seismic vulnerability assessment procedures – is brought about.

CHAPTER 2 Research Objectives and Scope

The objectives of this research are stated as following:

- To provide general overviews and critical comparisons of alternative assessment procedures from NZSEE 2006, ASCE 41-13, EN1998-3: 2005, NTC 2008, ATC 40, ATC 50, DBELA (see Chapter 3, Appendix A4)
- To provide general overviews and critical comparisons of the adopted analysis approaches in assessment, e.g. Linear Static Analysis (i.e. Linear Elastic Analysis, LSP, LEA), Linear Dynamic Analysis (i.e. LDP), Simple Lateral Mechanism Analysis (i.e. SLaMa), Nonlinear Static Analysis (i.e. Lateral Pushover Analysis, NSP, LPA), and Nonlinear Dynamic Analysis (i.e. Time History Analysis, NDP) (see Chapter 4, Appendix A5)
- To propose improvements and modifications to the current NZSEE 2006 Guidelines, especially to simplified analytical approaches (see Chapter 5, and Appendix A1, A2, A3)
- To provide a study of Christchurch reinforced concrete buildings (see Chapter 6)
- To evaluate the capability of simplified analytical approaches by: (1) comparison of the outputs from simplified approaches and more comprehensive approaches (i.e. numerical modelling); (2) correlation between the outputs from simplified approaches and the observed damages; (3) parametric study (see Chapter 7, 8, 9)
- To indicate the use of this research, and the parts requiring future researches and investigations

The scope of thesis is limited to reinforced concrete buildings only. The main focus is given to the frame-dominated type of structure, and only brief discussions regarding assessing shear walls are included. It is also worth noting that this thesis centres upon assessing capacities of reinforced concrete structures, and does not does not include elaborate research in assessing seismic demand.

CHAPTER 3 Overviews and Critical Comparison ofAssessment Procedures

3.1. Introduction

Among the various assessment procedures, simple or complicated, some of them have been codified with regulated steps and provided as standards or guidelines for engineers, while some are not clearly specified. It is vital to explore all these procedures in order to clearly define the features (together with advantages and disadvantages) of each procedure before making any suggestions to the current New Zealand assessment guidelines. It is also very essential to find out if different simplified assessment procedures are recommended.

In this chapter, general reviews and critical comparison of the codified assessment procedures applied in New Zealand, the USA and European countries (or particularly in Italy) are presented. Apart from the above mentioned codified procedures, alternative assessment procedures, such as Capacity Spectrum Method, Simplified Deformation-based Probabilistic Assessment Procedure and so on, are briefly introduced.

Table 3- 1: Summary of seismic assessment procedures (including the adopted analysis approaches)

Code Provisions	Less Detailed Assessment Level	Detailed Assessment Level
NZSEE 2006	Initial Seismic Assessment (ISA)	Detailed Seismic Assessment (DSA) Adopt Linear Elastic Analysis, Simple Lateral Mechanism Analysis or Lateral Pushover Analysis (i.e. LEA, SLaMa or LPA)
ASCE 41-13	Tier 1 Screening Tier 2 Deficiency-Based Evaluation Adopt Simplified Linear Static Analysis, Linear Static Analysis or Linear Dynamic Analysis (i.e. Simplified LSP, LSP or LDP)	Tier 3 Systematic Evaluation Adopt Linear Static Analysis, Linear Dynamic Analysis, Nonlinear Static Analysis or Nonlinear Dynamic Analysis (i.e. LSP, LDP, NSP, or NDP)
EN 1998-3: 2005	Knowledge Level 1 Adopt Linear Static Analysis or Linear Dynamic Analysis (LSP or LDP)	Knowledge Level 2 Knowledge Level 3 Adopt Linear Static Analysis, Linear Dynamic Analysis, Nonlinear Static Analysis, or Nonlinear Dynamic Analysis (i.e. LSP, LDP, NSP or NDP)
NTC 2008	Knowledge Level 1 Adopt Linear Static Analysis or Linear Dynamic Analysis (i.e. LSP or LDP)	Knowledge Level 2 Knowledge Level 3 Adopt Linear Static Analysis, Linear Dynamic Analysis, Nonlinear Static Analysis, or Nonlinear Dynamic Analysis (i.e. LSP, LDP, NSP or NDP)
ATC 40		Capacity Spectrum Method
ATC 58		PACT
	Displacement-Based Earthquake Loss Assessment Procedure (DBELA)	

3.2. Codified Assessment Procedures

The up-to-date assessment standards or guidelines that are applied in New Zealand, the USA, Europe and Italy are listed as following:

- NZSEE 2006 Guidelines – Assessment and Improvement of the Structural Performance of Buildings in Earthquakes
- ASCE 41-13 – Seismic Evaluation and Retrofit of Existing Buildings
- EN 1998-3: 2005 – Assessment and Retrofitting of Buildings
- NTC 2008 – Approvazione delle nuove norme tecniche per le costruzioni (Approval of New Technical Standards for Buildings)

Section 3.2.1 to 3.2.4 give concise reviews of the assessment procedures stated as above. Critical comparisons of these codified assessment procedures are made, and the main differences are discussed in Section 3.3.

3.2.1. NZSEE 2006 Guidelines (New Zealand Guidelines)

The assessment guidelines currently applied in New Zealand – Assessment and Improvement of the Structural Performance of Buildings in Earthquake – provided by New Zealand Society for Earthquake Engineering, were developed based on the previous documents, NZSEE 1996 Green Book and NZSEE 2000, and the Guidelines draw together New Zealand and international knowledge till the time when they were issued. The Guidelines are used by engineers and practitioners to predict or assess the response of a building in a seismic event, thereby, to strengthen or retrofit the building based on the results obtained from the assessment. NZSEE Section Revision on Section 3 Initial Seismic Assessment was released in 2013, introducing more detailed guidelines for Initial Seismic Assessment and adding preliminary assessment procedures for masonry structures. It has been proposed that revision and modification work of other sections of the current guidelines, particularly Section 7 Detailed Assessment of Reinforced Concrete Structures, will be accomplished in 2015. In Figure 3- 1, a summary of development of the New Zealand guidelines in history is presented.

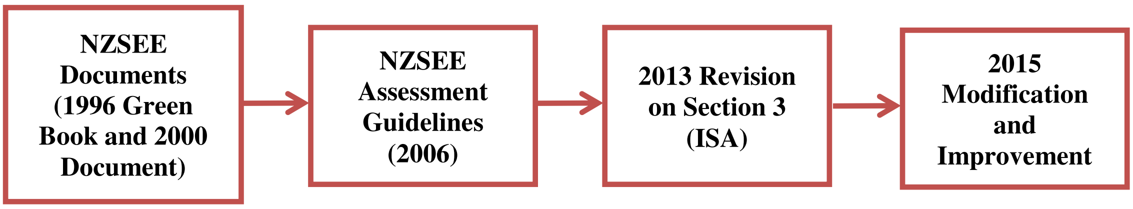


Figure 3- 1: Timeline of development of NZSEE 2006 Guidelines

In NZSEE 2006, a two-stage assessment process is recommended, shown as following:

- Initial Seismic Assessment – ISA
- Detailed Seismic Assessment – DSA

ISA, commenced as the first stage of assessment process, can be applied targeting either individual building or a large building inventory with limited available information. At the ISA stage, only the fundamental building information, such as building age, importance level, location, soil type, etc. is required as inputs to carry out Preliminary Screening, Prioritisation Process, and Initial Evaluation Procedure (i.e. IEP). To conduct an ISA, no specific structural analysis is required, and only simple calculations and engineering decisions are adopted. ISA can provide identification of Critical Structural Weaknesses (CSWs) of the buildings under interest, and can also compute Percentage of New Building Standard (%NBS) which is used as important indicate to determine potential Earthquake Prone Buildings (EPB) and the need of further assessment.

DSA, performed as the secondary stage of assessment process, is required for the buildings assessed as EPBs by ISA. However, it is worth recognising that DSA is also recommended for non-EPBs, especially for the buildings assessed as Earthquake Risk Buildings (ERB) by ISA. At the DSA stage, more detailed information, especially associated with the material and component properties and strengths, is needed as inputs to conduct structural analyses. Five analysis approaches are specified in the Guidelines, and the choice of analysis should depend on the level of sophistication required for the assessment considering objective of assessment, access of information and resources, quality of information, etc. Procedures for two linear approaches –Equivalent Static Analysis (i.e. LSA or LSP) and Modal Response Spectrum Analysis (i.e. LDP), and three nonlinear approaches – Simple Lateral Mechanism Analysis (i.e. SLaMa), Lateral Pushover Analysis (i.e. LPA or NSP) and Inelastic Time History Analysis (i.e. THA or NDP), are specified in NZSEE 2006 Guidelines Appendix 4E. Then the lateral seismic force and displacement capacity determined from the selected structural analysis should be compared to the demand computed by either force-based or displacement-based procedure. DSA can provide confirmation of CSWs determined by ISA, identification of CSWs not found in ISA, and also refinement of %NBS.

Section 3.2.1.1 and Section 3.2.1.2 give concise reviews of ISA and DSA, respectively, and the step-by-step procedures of ISA and DSA are presented in Appendix A4. The review and comparison of the analysis approaches adopted in assessment are presented in Chapter 4, and future improvements of NZSEE 2006 Guidelines are suggested in Chapter 6.

3.2.1.1. Initial Seismic Assessment (ISA)

As stated previously, to carry out an ISA, less information is required to determine inputs, and the critical inputs are listed as following:

- Building age (or year of design, or construction)
- Building importance level determined based on AS/NZS1170.0
- Construction type
- Any previous strengthening and to what standard if known
- Location of building
- Local soil type
- Number of storeys and estimated building height, in order to approximate fundamental period

If multiple buildings (or a large building inventory) are considered, before initiating Preliminary Screening Process or Initial Evaluation Procedure, a Prioritisation Process should be carried out in order to identify the buildings with potentially higher risk to life and the buildings of higher importance to the community in the aftermath of a severe seismic event.

Base on the information collected, for each criterion (e.g. age, importance level, construction type, seismic hazard, location, previous strengthening), values of prioritisation rating factors can be assigned, as specified in Table 3- 2. By ranking the buildings according to the product of the determined rating factors, the buildings of higher priority that require formal initial seismic assessment can be identified. More details concerning the prioritisation procedure are shown in Appendix A4.

Table 3- 2: Prioritisation rating factor (NZSEE 2006 Guidelines, 2013 Revision on Section 3, Table 3.1)

Criteria	Rating Factor			
	1	2	3	4
Age (when built or strengthening code)	Pre 1965	1965 - 1975	1976 - 1992	Post 1992
Importance Level	IL 4	IL 3	IL 2	IL 1
Construction Type	URM Bearing Wall	Timber Framed on side of hill	Other	Light Timber Framed on Flat Site
Seismic Hazard Factor[1]	$\geq$0.4	0.31 – 0.39	0.14 - 0.3	$\leq$0.13
Location[2]	On critical transportation route	In CBD	Other	
Previous Strengthening	Add 1 to rating factor for age or multiply Rating factor for age by % strengthening, whichever gives the lowest value			

1. Seismic hazard factor differentiation may be relevant for consideration of a property portfolio across New Zealand.
2. Rating factor = 4 if unlikely to have any effect on neighbouring property or streets, ie. location unimportant.

Figure 3- 2 is taken from NZSEE 2006 Guidelines (2013 Revision on Section 3) to illustrate the overall process of ISA. In sections 3.2.1.1.1 and 3.2.1.1.2, general reviews of Preliminary Screening and Initial Evaluation Procedure are presented, with more details shown in Appendix A4.

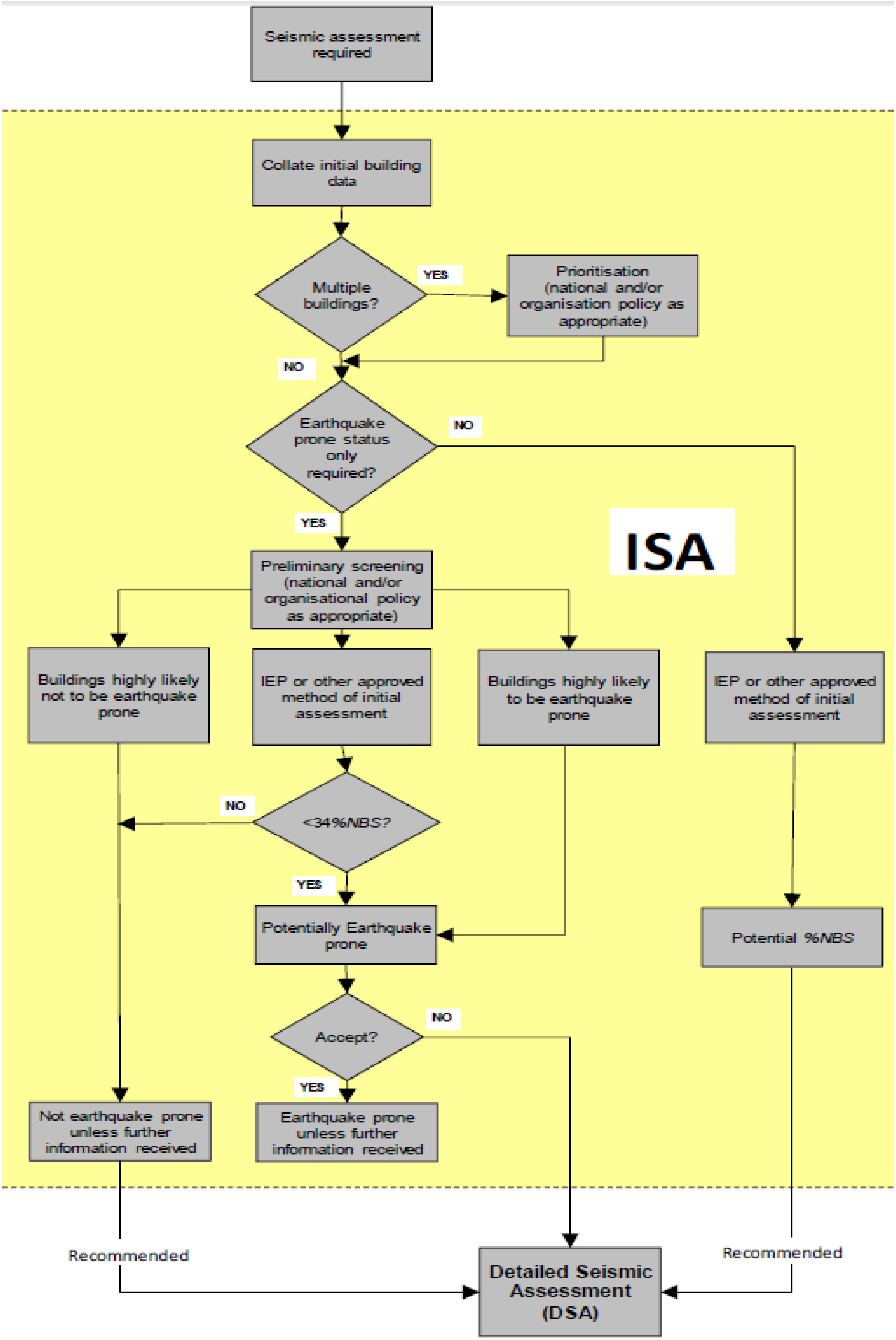


Figure 3- 2: Diagrammatic representation of Initial Seismic Assessment process (NZSEE 2006 Guidelines, 2013 Revision on Section 3 Figure 3.1)

3.2.1.1.1. Preliminary Screening

Based on past experience and without entering into formal assessment process, screening of structures can be performed to preliminary identify potentially EPBs. By only applying Preliminary Screening, unreinforced masonry buildings without previous strengthening can be decided as potentially EPBs. Also only by Preliminary Screening, timber framed structures without heavy roofs and located on flat sites with less than 600mm height of ground floor, and post 1976 buildings of importance level 1-3, can be decided as potentially not EPBs. To assess other types of structures, more sophisticated assessment procedures are required.

It is worth noting that even if a building is assessed to be non-EPB by Preliminary Screening, the possibility of the existence of CSWs still needs to be considered, as the existence of CSWs may lead to a sudden, non-ductile partial or global failure. For examples, in some post 1976 buildings, structural deficiencies may exist due to complex configurations, transfer structured systems, offset columns, inadequate components (e.g. diaphragms, infilled walls, stairs, bracings, etc.) and insufficient ductility detailing.

3.2.1.1.2. Initial Evaluation Procedure

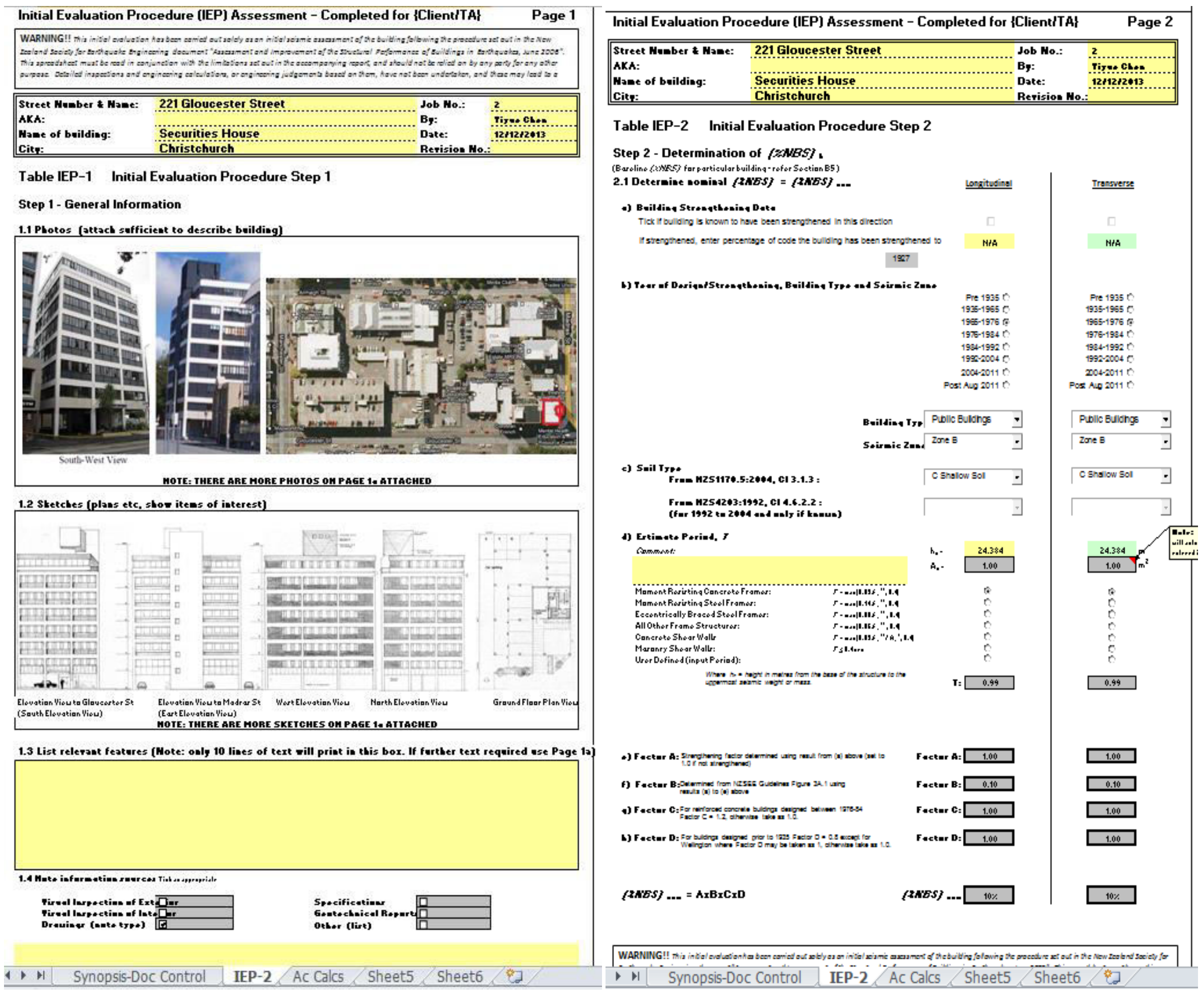


Figure 3- 3: Excel spreadsheet of IEP (Page 1-2) (NZSEE2006, 2013 Revision on Section 3)

Initial Evaluation Procedure (IEP) is adopted in NZSEE 2006 Guidelines (2013 Revision on Section 3) as the formal initial seismic evaluation procedure. IEP identifies potential CSWs, and determines EPB based on the calculated %NBS values. The excel spreadsheets with detailed IEP steps and calculations are shown in Figure 3- 3, and a flowchart of the procedure is provided as Figure 3- 4.

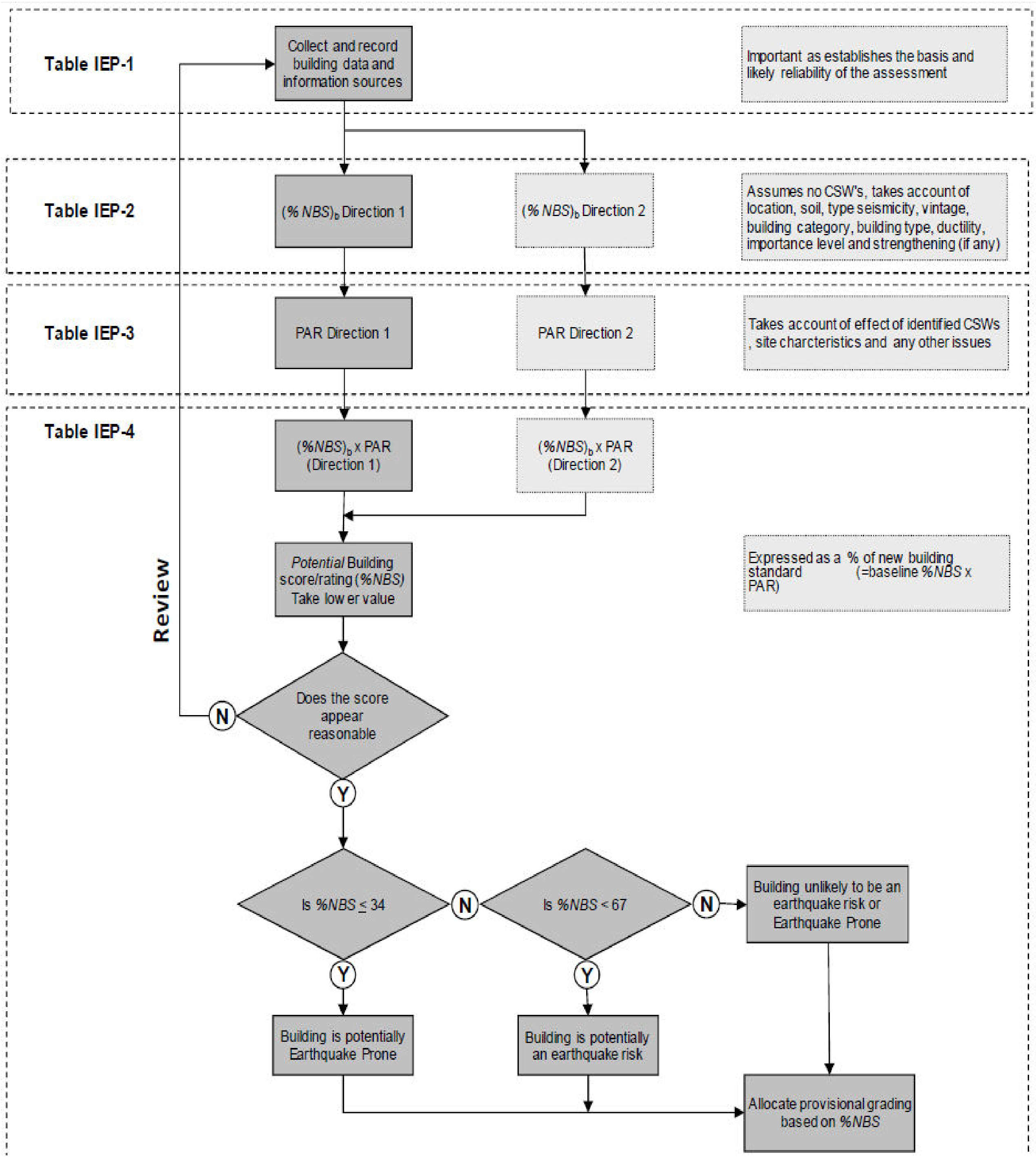


Figure 3- 4: Diagrammatic representation of Initial Evaluation Procedure (NZSEE 2006, 2013 Revision on Section 3, Figure 3.2)

The critical steps of IEP are summarised in the following paragraphs, and more details can be found in Appendix A4. The limitations of applying IEP are discussed in the end of this section.

- Collect general information
- Determine baseline percentage of new building standard $(\%NBS)_b$:

$$(\%\mathrm{NBS})_{\mathrm{nom}} = \mathrm{A} \times \mathrm{B} \times \mathrm{C} \times \mathrm{D}$$

$$(\%\mathrm{NBS})_{\mathrm{b}} = (\%\mathrm{NBS})_{\mathrm{nom}} \times \mathrm{E} \times \mathrm{F} \times \mathrm{G} \times \mathrm{H} \times \mathrm{I}$$

$$= (\%\mathrm{NBS})_{\mathrm{nom}} \times \frac{1}{\mathrm{N(T,D)}} \times \left(\frac{1}{\mathrm{Z}} \text{ or } \frac{\mathrm{Z}_{1992}}{\mathrm{Z}}\right) \times \frac{\mathrm{IR}_0}{\mathrm{R}} \times (\mu \text{ or } 1) \times \frac{1}{\mathrm{S_p}}$$

A~I are factors associated with strengthening, year of design, soil type, fundamental period, near fault effect, hazard, return period, ductility, and structural performance. The determination of these factors are seen in NZSEE 2006 Guidelines (2013 Revision on Section 3) and the IEP Excel Spreadsheet. Table 3- 3, Table 3- 4, Table 3- 5and Figure 3- 5provide guidance regarding the determination of current return period factor, ductility factor and performance factor.

Table 3- 3: Return period factor, R (NZSEE 2006 Guidelines, 2013 Revision on Section 3 Table 3A.1)

Importance Level	Comments	R
1	Structures presenting a low degree of hazard to life and other property	0.5
2, or if otherwise unknown	Normal structures and structures not in other importance levels	1.0
3	Structures that as a whole may contain people in crowds or contents of high value to the community or pose risks to people in crowds	1.3
4	Structures with special post-disaster functions	1.8
5	Special structures (outside the scope of this Standard—acceptable probability of failure to be determined by special study)	

Table 3- 4: Maximum ductility factors in IEP (NZSEE 2006 Guidelines, 2013 Revision on Section 3 Table 3A.2)

Structure Type	Maximum ductility factor			
	Pre-1935	1935-65	1965-76	>1976
Unstrengthened URM buildings	1.5	1.5	N/A	N/A
All other buildings	2	2	2	6

Table 3- 5: Ductility scaling factor, H (NZSEE 2006 Guidelines, 2013 Revision on Section 3 Table 3A.3)

	Structural Ductility Factor, μ							
	1.0		1.25		1.50		2	
Soil Type	A,B,C & D	E	A,B,C & D	E	A,B,C & D	E	A,B,C & D	E
Period, *T*								
≤ 0.40s	1	1	1.14	1.25	1.29	1.50	1.57	1.70
0.50s	1	1	1.18	1.25	1.36	1.50	1.71	1.75
0.60s	1	1	1.21	1.25	1.43	1.50	1.86	1.80
0.70s	1	1	1.25	1.25	1.50	1.50	2.00	1.85
0.80s	1	1	1.25	1.25	1.50	1.50	2.00	1.90
≥1.00s	1	1	1.25	1.25	1.50	1.50	2.00	2.00

Note:
For buildings designed post 1976, Factor H shall be taken as 1.0.

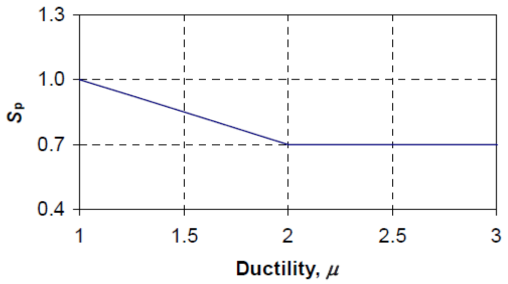


Figure 3- 5: Structural performance factor, S_p (NZSEE 2006 Guidelines2013 Revision on Section 3 Figure 3A.2)

- Determine performance achievement ratio (PAR)

$$PAR = A \times B \times C \times D \times E \times F$$

A~F are factors associated with potential CSWs, e.g. plan and vertical irregularity, short columns, pounding, height difference, site characteristics, etc.

Table 3- 6: Guide to severity of CSWs (NZSEE 2006 Guidelines, 2013 Revision on Section 3 Table 3A.4)

	Effect on structural performance		
Critical structural weakness	**Severe**	**Significant**	**Insignificant**
Plan irregularity			
L-shape, T-shape, E-shape	Two or more wings length/ width > 3.0, or one wing length/width >4	One wing length/width > 3.0	All wings length/width ≤ 3.0
Long narrow building where spacing of lateral load resisting elements is …	> 4 times building width	> 2 times building width	≤ 2.0 times building width
Torsion (Corner Building)	Mass to centre of rigidity offset > 0.5 width	Mass to centre of rigidity offset > 0.3 width	Mass to centre of rigidity offset ≤ 0.3 width or effective torsional resistance available from elements orientated perpendicularly.
Ramps, stairs, walls, stiff partitions	Clearly grouped, clearly an influence	Apparent collective influence	No or slight influence
Vertical irregularity			
Soft storey	Lateral stiffness of any storey < 0.7 of lateral stiffness of any adjoining storeys	Lateral stiffness of any storey < 0.9 of lateral stiffness of the adjoining storeys	Lateral stiffness of any storey ≥ 0.9 of lateral stiffness of the adjoining storeys
Mass variation	Mass of any storey < 0.7 of mass of adjoining storey	Mass of any storey < 0.9 of mass of adjoining storey	Mass of any storey ≥ 0.9 of mass of adjoining storey
Vertical discontinuity	Any element contributing > 0.3 of the stiffness/strength of the lateral force resisting system discontinues vertically	Any element contributing > 0.1 of the stiffness/strength of the lateral force resisting system discontinues vertically	Only elements contributing ≤0.1 of the stiffness/strength of the lateral force resisting systems are discontinuous vertically
Short columns Columns < 70% storey height between floors clear of confining infill, beams or spandrels	Either > 80% short columns in any one side Or > 80% short columns in any storey	> 60% short columns in any one side > 60% columns in any one storey	No, or only isolated, short columns, or Columns with width > 1.2 m, or Free column height/ column width ≥2.5.
Pounding effect Vertical differences between floors > 20% storey height of building under consideration	0 < separation < 0.005 *H* where *H* = height to the level of the floor being considered	0.005 *H* < separation < 0.01 *H*	Separation > 0.01 *H* Floors aligning and height difference is one storey
Height difference effect No adjacent building, or height difference < 2 storeys			No penalty
Height difference > 4 storeys	0 < separation < 0.005 *H* where *H* = height of the lower building and separation is measured at *H*	0.005 *H* < separation < 0.01 *H*	Separation > 0.01 *H*, or Floors aligning and height difference < 2 storeys, or At least one building is light weight construction
Site characteristics	Unstable site Extensive landslide from above or below Probable liquefaction	Potential for site instability Landslide from above or below Liquefaction potential	Not a significant threat

As shown in Table 3- 6, values of factors A to E are assigned depending on the severity of critical structural weaknesses, i.e. severe, significant and insignificant. A compensating factor F is also adopted to account for any parameters that have not been considered in previous evaluation, based on judgment of engineers. PAR should be calculated for each orthogonal direction.

- Determine the percentage of new building standard, %NBS

$$\%NBS = PAR \times (\%NBS)_b$$

- Determine if the building is EPB or ERB, and give provisional grading

  In Table 3- 7 and Table 3- 8, the determination of EPB or ERB, building grade and building risk based on %NBS values is summarised. More details concerning grading scheme and life-safety risk description are given in NZSEE2006.

Table 3- 7: Acceptance criteria of NZSEE 2006 Guidelines

%NBS	EPB or ERB	NOTE
%NBS < 34%	Potentially EPB, DSA required	Special care should be taken when %NBS = 30% ~ 34%
34% ≤ %NBS < 67%	Potentially ERB, DSA recommended	Special care should be taken when %NBS = 65% ~ 67%
%NBS ≥ 67%	No significant earthquake risk	%NBS ≥ 100%: Erroneous indication

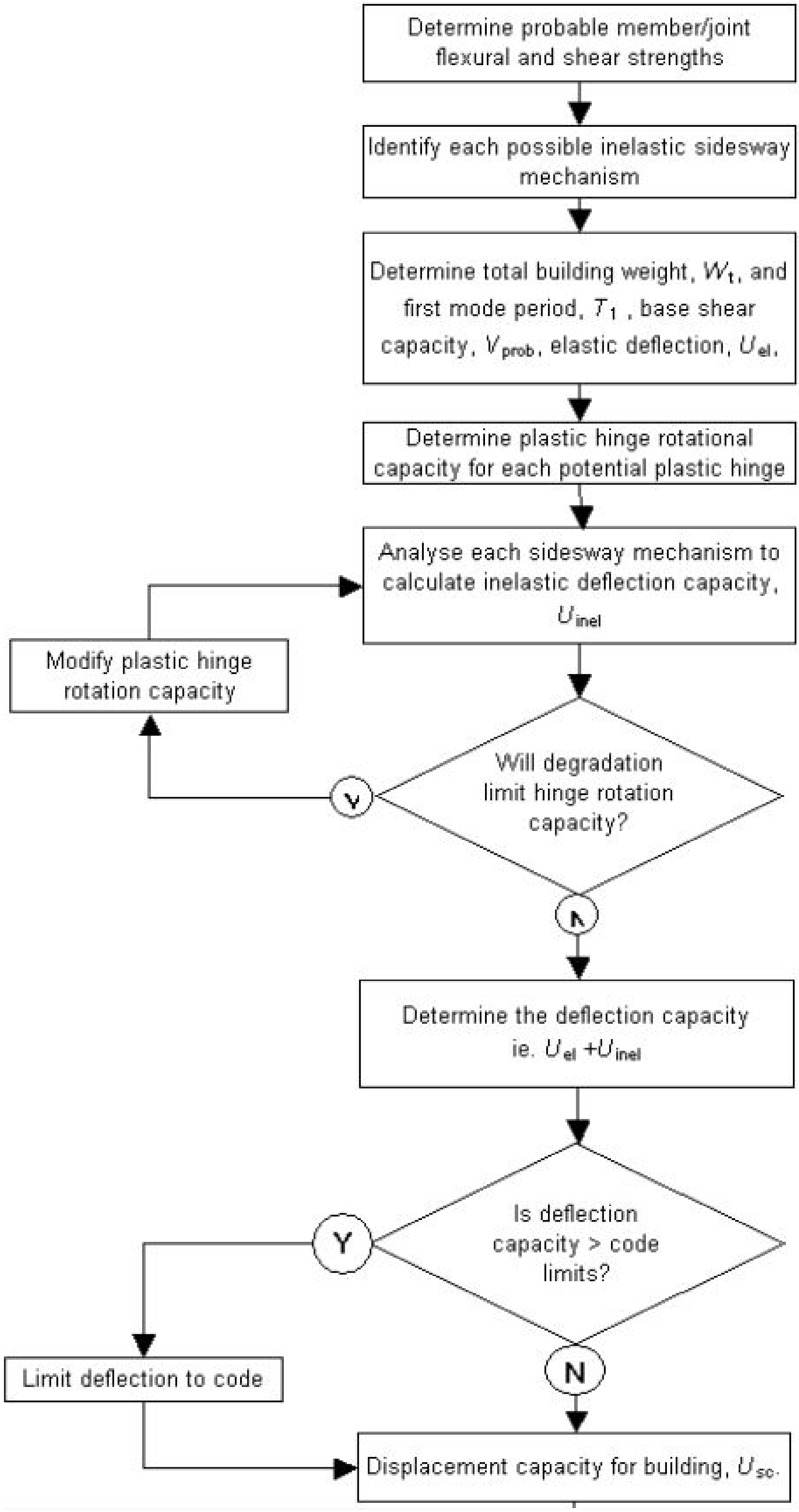
Table 3- 8: Relative earthquake risk (NZSEE 2006 Guidelines, 2013 Revision on Section 3, Template Covering Letter – Building Owner or Tenant Commissioned IEP Table 1)

Building Grade	Percentage of New Building Strength (%NBS)	Approx. Risk Relative to a New Building	Life-safety Risk Description
A+	>100	<1	Low Risk
A	80 to 100	1 to 2 times	Low Risk
B	67 to 79	2 to 5 times	Low or Medium Risk
C	34 to 66	5 to 10 times	Medium Risk
D	20 to 33	10 to 25 times	High Risk
E	<20	More than 25 times	Very High Risk

The limitations of IEP are recognised as:

- IEP results can be either conservative or vice versa, and may not be truly representative of seismic performance of the building due to unidentified design, construction issues, or unrecognised CSWs.
- Reliability of IEP depends on level of information and engineers' judgment.
- IEP is designed to assess the building against the ultimate limit state only.
- IEP does not account for seismic performance of non-structural items, nor possible detrimental effects of neighbouring building.

3.2.1.2. Detailed Seismic Assessment (DSA)



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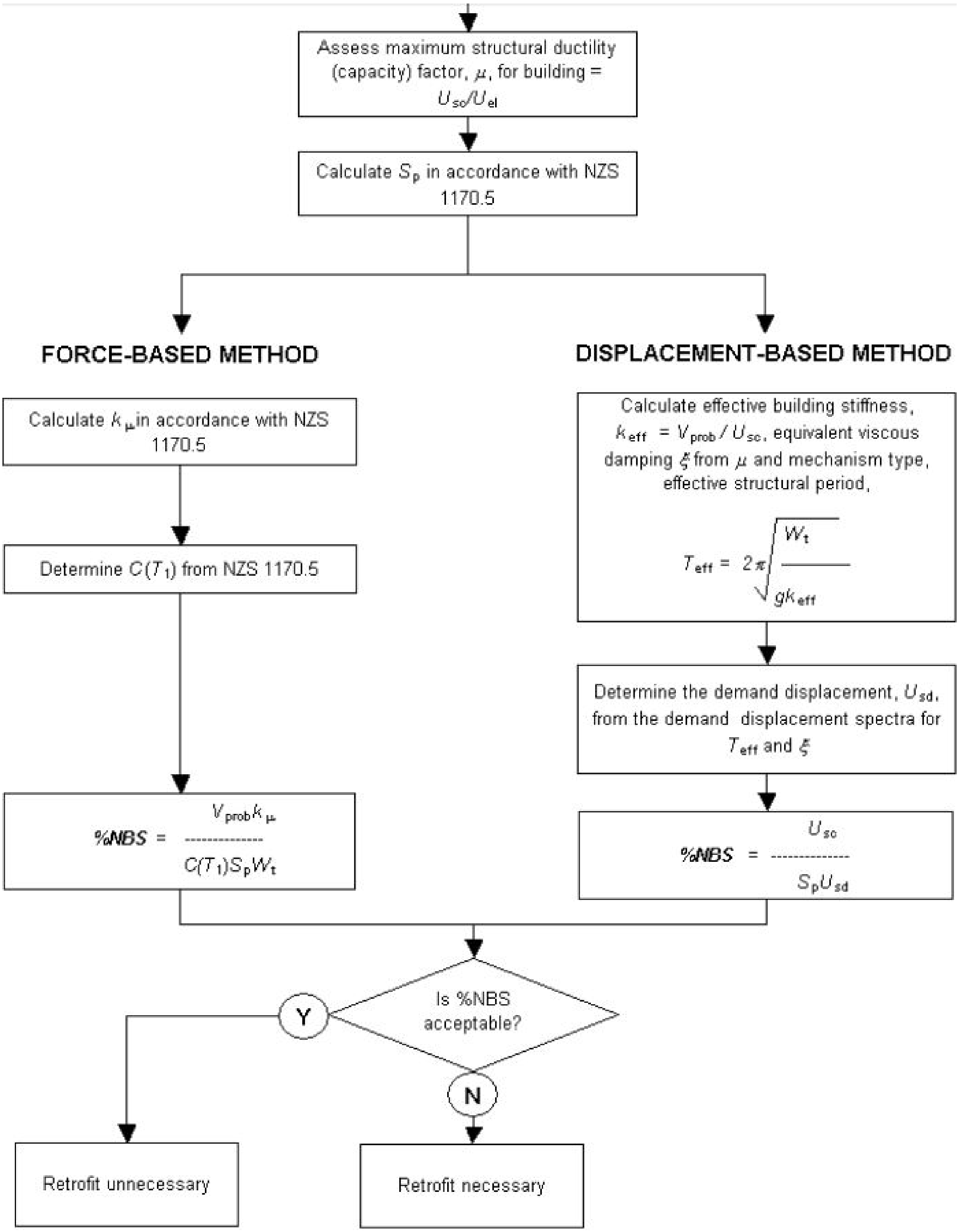
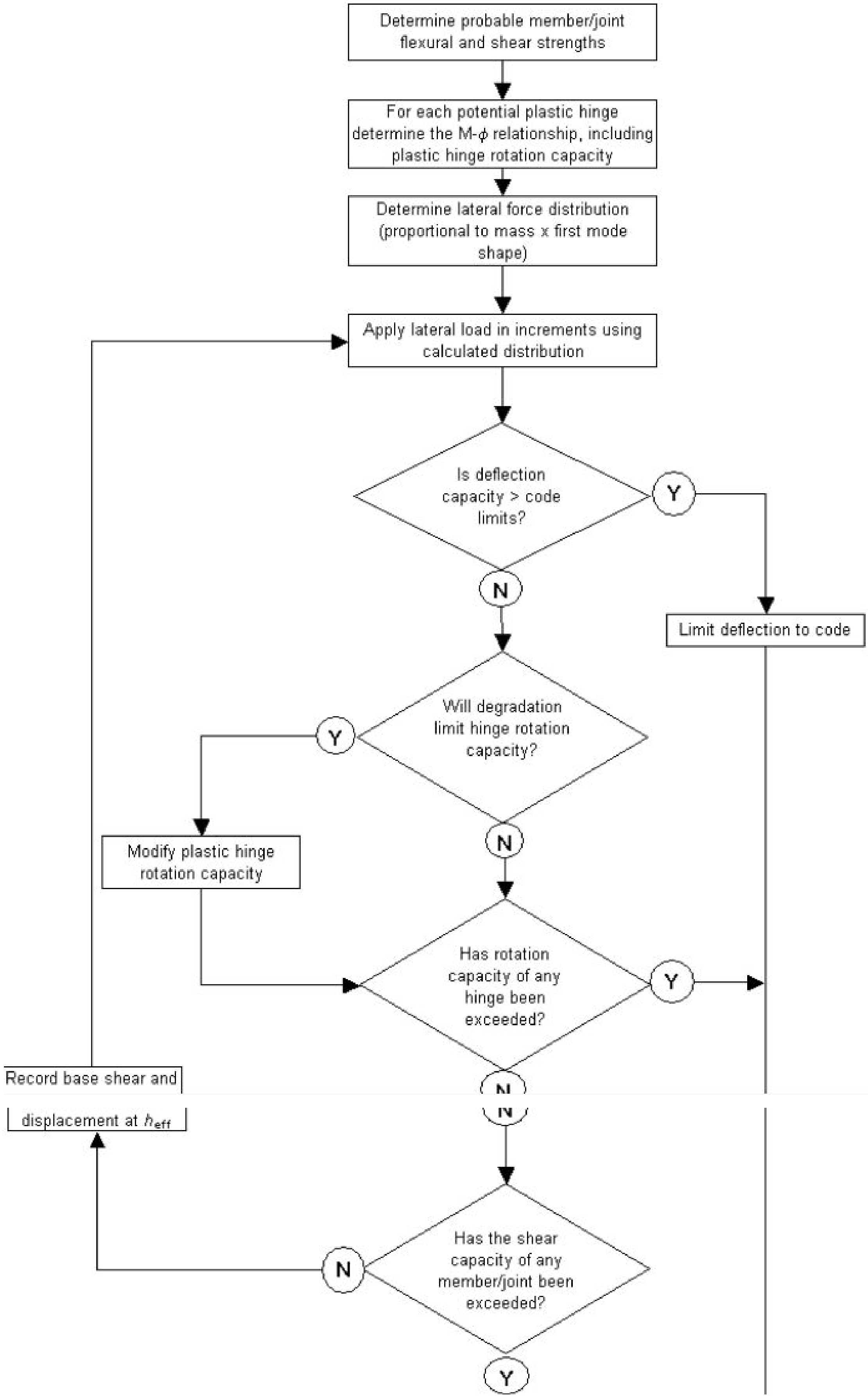


Figure 3- 6: Consolidated force/displacement based assessment procedure (with static analysis for each principal direction) (NZSEE 2006 Guidelines Figure 6.5)



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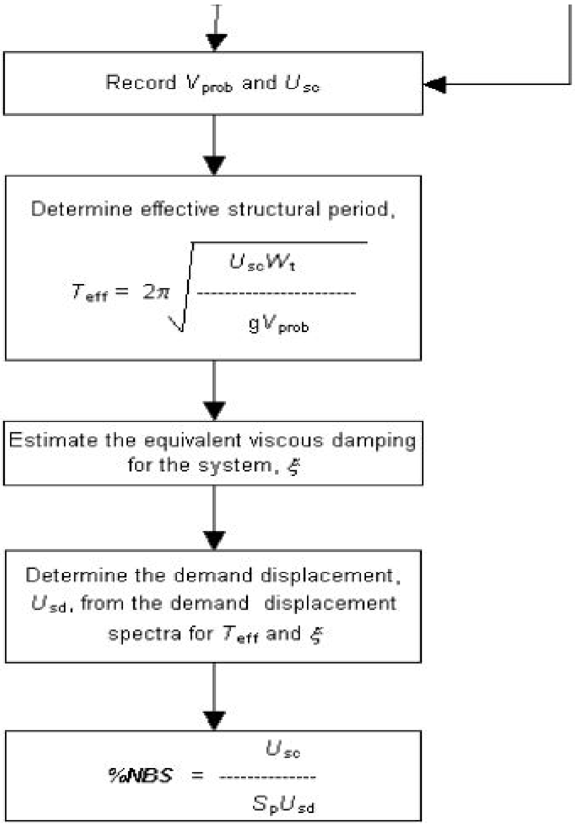


Figure 3- 7: Assessment procedure using nonlinear pushover analysis (NZSEE 2006 Guidelines Figure 6.6)

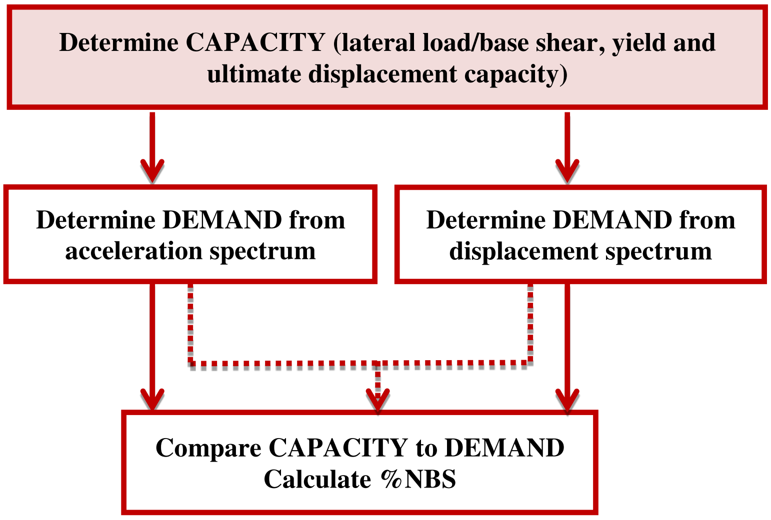


Figure 3- 8: Simplification of assessment procedures (the solid arrows indicate the progress of the current procedures presented in NZSEE 2006)

The consolidated force displacement assessment procedure with static analysis for each principal direction is shown in Figure 3- 6, and the procedure with nonlinear pushover analysis is shown in Figure 3- 7. Both figures are duplicated from NZSEE 2006 Guidelines. For the purpose of simplification, Figure 3- 8 provides a simpler flowchart, with the solid arrows indicating the currently applied processes, and the dashed arrows indicating the potential improvement. Most engineers are more familiar with the force-based procedure, despite the fact that the displacement-based procedure produces more rational and less conservative outcomes, as the predictive results are more straightforwardly correlated to the deformation of the components or the structure interstorey drifts. Therefore, it should be suggested that the outcomes from the force-based procedure should be cross-checked by carrying out the displacement-based procedure.

In both procedures (force-based and displacement-based) from NZSEE 2006, it has been found that the procedures associated with the determination of component capacities (flexural and shear), total building base shear capacity and displacement capacity (i.e. yield and ultimate displacement of the structure) are the same. The determination of component flexural and shear capacities, in NZSEE 2006, are specified with the consideration of slab contribution to beam strength, deterioration of bond due to cyclic loading, bond slip effect at lap splice, etc, according to New Zealand design code SNZ1995 (i.e. NZS3101:1995). In order to predict global response of the structure based on the assessment of components, three analysis approaches – Linear Elastic Analysis (including static and dynamic analyses), Simple Lateral Mechanism Analysis (based on the calculation of storey sway index and prediction of failure mechanism), and Nonlinear Lateral Pushover Analysis–are recommended. The choice of analysis approach depends on the level of sophistication required for the assessment considering objective of assessment, access and quality of information at material level and component level, availability of tools and programmes, and constraint of time and expense. With applying the analysis approach of the least to the most sophistication, capacity of the structure can be determined as only imprecise assumption to accurate approximation. With the determined ductility capacity, together with the ductility demand estimated from acceleration spectrum or displacement spectrum, %NBS can be calculated by comparing the capacity to the demand. More details regarding the step-by-step DSA procedure are shown in Appendix A4.

3.2.1.2.1. Material properties

Probable strengths of concrete and reinforcing steel should be applied in assessment, as required in NZSEE 2006 Guidelines. The guidelines not only define that material properties should be obtained from construction documents, surveys and physical testing of representative samples of materials, but also provide procedures to estimate material strengths in the absence of reliable information. The guidelines also give a very brief summary concerning the history of material strengths. More information regarding the determination of material properties is shown in Section 3.3.3.

3.2.1.2.2. Component flexural and shear capacities

As mentioned in previous, instructions to determine flexural and shear capacities of components, including beams, columns, joins, walls, etc. are provided on the basis of NZS301:1995 (i.e. SNZ 1995). Therefore, it is required that the current guidelines should be updated and improved based on the most advanced knowledge and research. Detailed procedures associated with the determination of component capacities are discussed in Section 3.3.4 and Section 4.6.

3.2.1.2.3. Determination of global mechanism and choice of analysis approaches

In NZSEE 2006 Guidelines Appendix 4E, five analysis approaches are specified:

- Equivalent Static Analysis (i.e. Linear Static Analysis, LSA or LSP)
- Modal Response Spectrum (i.e. Linear Dynamic Analysis, LDS or LDP)
- Simple Lateral Mechanism Analysis (SLaMa, i.e. analytical “by-hand” pushover analysis)
- Lateral Pushover Analysis (i.e. Nonlinear Static Analysis, LPA, NSA or NSP)
- Inelastic Time History Analysis (i.e. Nonlinear Dynamic Analysis, THA, NDA or NDP)

In DSA, three approaches from the above list are recommended to determine probable lateral seismic force capacity and displacement capacity, as stated in NZSEE 2006 Guidelines Section 7:

- Linear Elastic Analysis – a linear analysis, by determining the first hinge forming in the structure, gives a lower bound of probable lateral force capacity (or displacement capacity) of the structure.
- Simple Lateral Mechanism Analysis – an analytical (i.e. “by-hand”) pushover analysis, by determining the failure mechanism and computing a bilinear pushover curve, gives an upper bound of probable lateral force capacity (or displacement capacity) of the structure.
- Lateral Pushover Analysis – a numerical nonlinear analysis, by determining the sequence of plastic hinges forming and approximating the actual nonlinear structural response, gives the most accurate probable lateral force capacity (or displacement capacity) of the structure among the three approaches. The recommended analysis programme is RUAUMOKO.

The choice of a proper analysis approach in seismic assessment is vital, and details regarding this issue are shown in Chapter 9. General reviews of all analysis approaches, along with discussion on the differences of the same type of analyses in different codified assessment procedures are presented in Chapter 4.

3.2.1.2.4. Determination of demand

Seismic demand can be determined from either acceleration spectrum (in force-based procedure) or from displacement spectrum (in displacement-based procedure). Detailed procedures to determine demands (acceleration or displacement) are shown in Appendix A4.

3.2.2. ASCE 41-13 (American Code)

ASCE 41-13, combining the three-tiered assessment procedure in ASCE 31-03 and the technical provisions for analytical approaches in ASCE 41-06, presents a new state of the practice in seismic evaluation and retrofit of existing buildings. Figure 3- 9 illustrates the development of seismic evaluation regulations since 1970s, along with the timeline for the issue of retrofit or rehabilitation standards, pre-standards and technical reports. Before the first document of seismic assessment (ATC 14) came into act, seismic evaluation was left solely to the judgment of the engineers, and was carried out referring to the new building design. During 1990s, seismic assessment and rehabilitation had been dramatically developed, and much more detailed procedures and analysis methods were specified in separated documents. ASCE 41-13, however, were developed to target combining seismic assessment and retrofit into one document. It has not only brought consistency to assessment and retrofit processes, but also incorporated many technical advances occurred in past years together with lessons learned from many recent earthquakes.

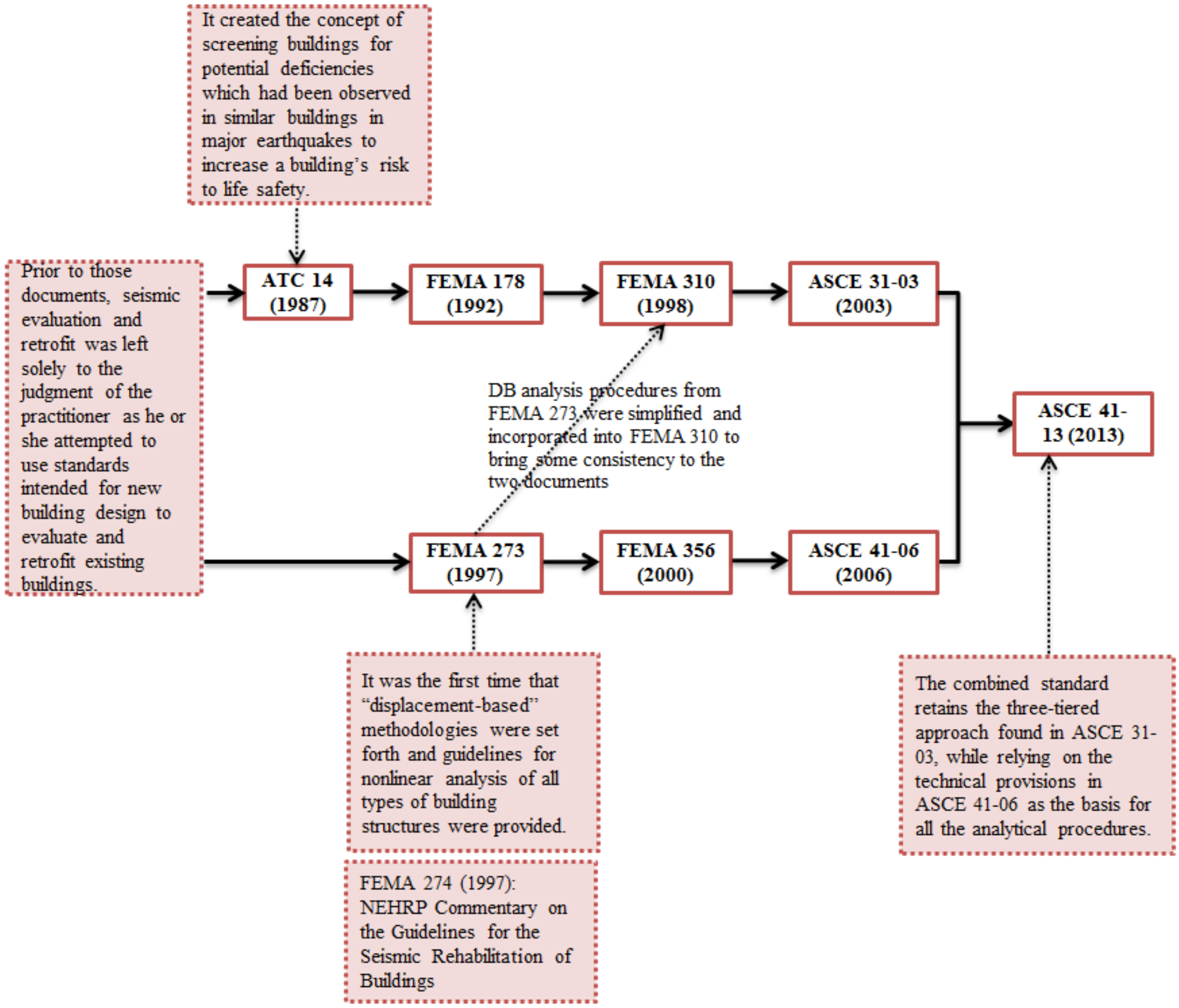
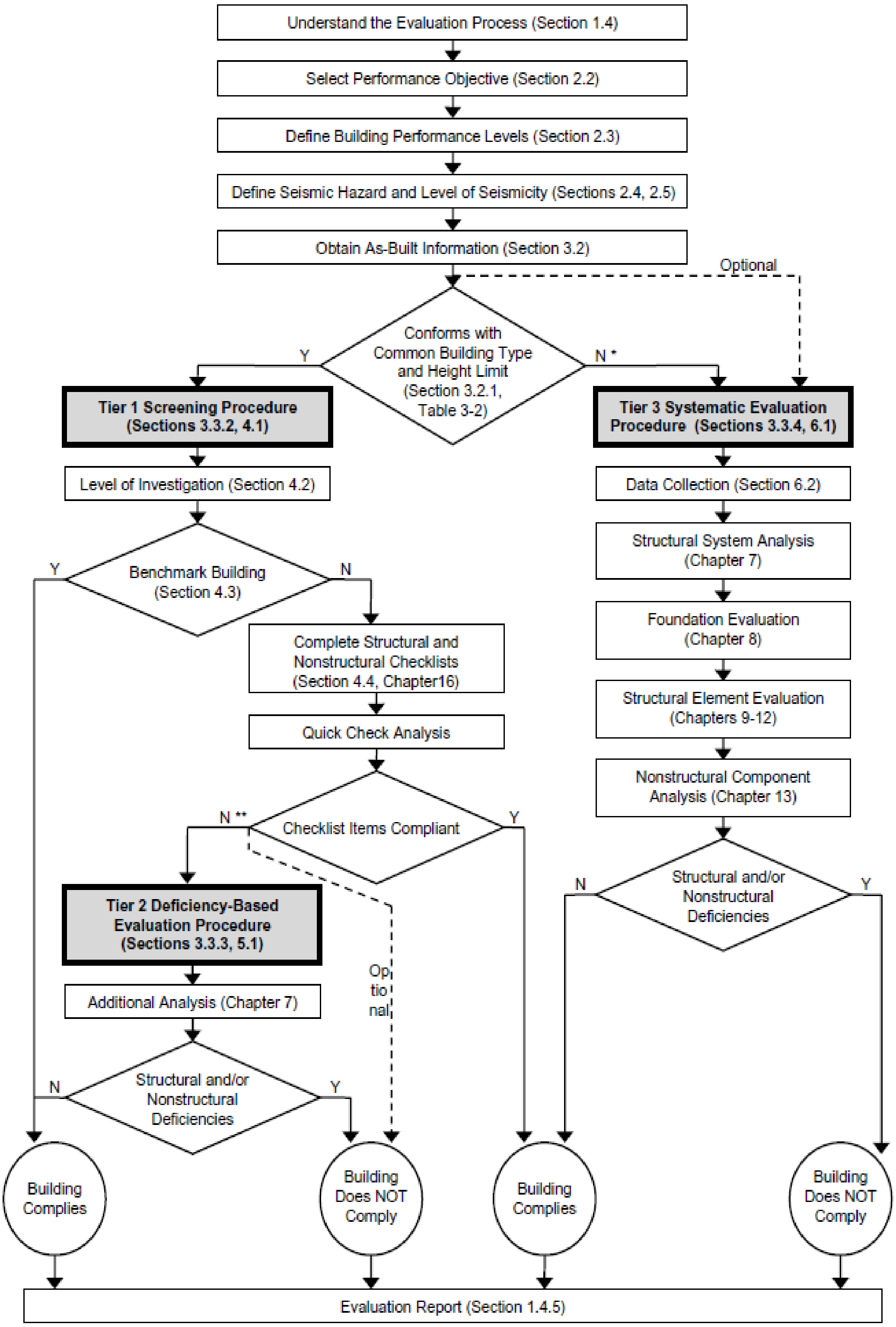


Figure 3- 9: Timeline for the development of seismic evaluation, retrofit or rehabilitation standards, pre-standards or technical reports

As stated in previous, the three-tiered assessment procedure from ASCE 31-01 is retained in ASCE 41-13, with modifications made to each tier. Figure 3- 10 illustrates the overall assessment process with the three tiers highlighted, and Figure 3- 11 shows a simplification of the process.



* It may be beneficial for the engineer to perform a Tier 1 Screening Evaluation prior to a Tier 3 Systematic Evaluation even though it is not required.
** The evaluation process may proceed directly to the Tier 3 Systematic Evaluation as an option.

Figure 3- 10: Evaluation process (ASCE 41-13 FIG. C1-1 Evaluation Process)

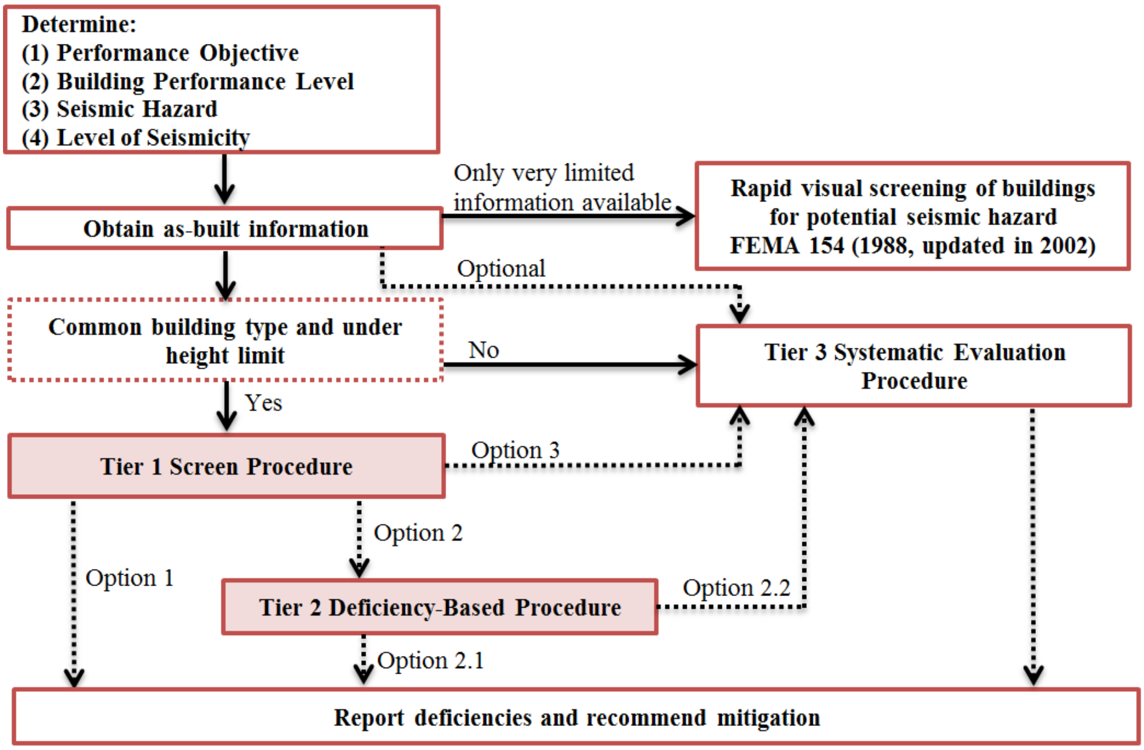


Figure 3- 11: Simplified evaluation process flowchart

Tier 1 Evaluation, requiring minimum level of information as inputs, involves only a screening process of buildings with completion of a series of checklists of building configuration, structural components, nonstructural components, foundation, geological hazard, site conditions, etc. The Tier 1 evaluation aims to check compliance with provisions specified in this standard (i.e. ASCE 41-13) and identify potential structural deficiencies. At this stage, only minimum level of analysis is needed, for instance, a simplified linear static procedure with simple calculations of component strength or stiffness.

Tier 2 Deficiency-based Evaluation, targeting to confirm the structural deficiencies identified in Tier 1 or to demonstrate adequacy of the structure, involves more sophisticated analyses and evaluation process compared to Tier 1 evaluation. If the Tier 2 evaluation procedure demonstrates the adequacy of the structure with respect to all of the “Noncompliant” or “Unknown” statements (i.e. the structural deficiencies identified) in the Tier 1 evaluation, then the building can be proved to be complied with the selected Performance Objective and further evaluation may not be required. However, if the deficiencies identified in Tier 1 are confirmed, engineers may choose to either conclude the evaluation then report the deficiencies, or proceed to Tier 3 to carry out a more comprehensive, systematic seismic assessment.

It is worth noting that Tier 1 and Tier 2 evaluation permits to demonstrate compliance with performance objectives for the structural performance levels S1 (IO-Immediate Occupancy), S3 (LS-Life Safety) and non-structural performance levels NB (PS-Position Retention), NC (LS-Life Safety).

The target building performance levels and the basic performance objectives for the existing buildings are defined in Table 3- 9 and Table 3- 10, retrieved from Table C2-8 and Table 2-1 in ASCE 41-13 Chapter 2.

Table 3- 9: Target building performance levels (ASCE41-13 Table C2-8)

Nonstructural Performance Levels	Structural Performance Levels: Immediate Occupancy (S-1)	Damage Control (S-2)	Life Safety (S-3)	Limited Safety (S-4)	Collapse Prevention (S-5)	Not Considered (S-6)
Operational (N-A)	Operational 1-A	2-A	NR[a]	NR[a]	NR[a]	NR[a]
Position Retention (N-B)	Immediate Occupancy 1-B	2-B	3-B	4-B	NR[a]	NR[a]
Life Safety (N-C)	1-C	2-C	Life Safety 3-C	4-C	5-C	6-C
Not Considered (N-D)	NR[a]	NR[a]	3-D	4-D	Collapse Prevention 5-D	No evaluation or retrofit

NOTE: NR = Not recommended.
[a]Combining low Structural Performance Level with high Nonstructural Performance Level, or the converse, is not recommended for several reasons. For example, having a low Structural Performance Level may lead to damage that prohibits actually achieving the desired Nonstructural Performance Level regardless of whether the nonstructural elements were retrofit to meet that Performance Level. Additionally, not addressing nonstructural hazards when a higher Structural Performance Level retrofit is undertaken may lead to an unbalanced design, where life safety hazards caused by nonstructural items are still present.

Table 3- 10: Basic Performance Objective for Existing Buildings – BPOE (ASCE41-13 Table 2-1)

Risk Category	Tier 1[a] BSE-1E	Tier 2[a] BSE-1E	Tier 3 BSE-1E	Tier 3 BSE-2E
I & II	Life Safety Structural Performance Life Safety Nonstructural Performance (3-C)	Life Safety Structural Performance Life Safety Nonstructural Performance (3-C)	Life Safety Structural Performance Life Safety Nonstructural Performance (3-C)	Collapse Prevention Structural Performance Nonstructural Performance Not Considered (5-D)
III	See footnote *b* for Structural Performance Position Retention Nonstructural Performance (2-B)	Damage Control Structural Performance Position Retention Nonstructural Performance (2-B)	Damage Control Structural Performance Position Retention Nonstructural Performance (2-B)	Limited Safety Structural Performance Nonstructural Performance Not Considered (4-D)
IV	Immediate Occupancy Structural Performance Position Retention Nonstructural Performance (1-B)	Immediate Occupancy Structural Performance Position Retention Nonstructural Performance (1-B)	Immediate Occupancy Structural Performance Position Retention Nonstructural Performance (1-B)	Life Safety Structural Performance Nonstructural Performance Not Considered (3-D)

[a]For Tier 1 and 2 assessments, seismic performance for the BSE-2E is not explicitly evaluated.
[b]For Risk Category III, the Tier 1 screening checklists shall be based on the Life Safety Performance Level (S-3), except that checklist statements using the Quick Check procedures of Section 4.5.3 shall be based on MS-factors and other limits that are an average of the values for Life Safety and Immediate Occupancy.

Also, Tier 1 and Tier 2 evaluation is intended for buildings meeting the criteria for the Common Building Type, as shown in Table 3- 11. If Tier 1 and Tier 2 evaluation procedures are permitted and are selected to apply, the process must begin with Tier 1, followed by Tier 2 as warranted, and Tier 3 evaluation may be used to further investigate into the deficiencies identified in previous tiers or to identify any other deficiencies that are missed. If Tier 1 and Tier 2 procedures are not permitted or engineers tend to choose a more sophisticated and comprehensive evaluation procedure, Tier 3 evaluation shall be conducted in accordance.

Table 3- 11: Building type limitations on the use of the Tier 1 and Tier 2 procedures (ASCE 41-13 Table 3-2)

	Number of Stories[b] beyond which the Tier 3 Systematic Procedures Are Required							
	Level of Seismicity							
	Very Low		Low		Moderate		High	
Common Building Type[a]	S-3	S-1	S-3	S-1	S-3	S-1	S-3	S-1
Wood Frames								
Light (W1)	NL	NL	NL	4	4	4	4	4
Multi-story, multi-unit residential (W1a)	NL	NL	NL	6	6	6	6	4
Commercial and industrial (W2)	NL	NL	NL	6	6	6	6	4
Steel Moment Frames								
Rigid diaphragm (S1)	NL	NL	NL	12	12	8	8	6
Flexible diaphragm (S1a)	NL	NL	NL	12	12	8	8	6
Steel Braced Frames								
Rigid diaphragm (S2)	NL	NL	NL	8	8	8	8	6
Flexible diaphragm (S2a)	NL	NL	NL	8	8	8	8	6
Steel Light Frames (S3)	NL	1	1	1	1	1	1	1
Dual Systems with Backup Steel Moment Frames (S4)	NL	NL	NL	12	12	8	8	6
Steel Frames with Infill Masonry Shear Walls								
Rigid diaphragm (S5)	NL	NL	NL	12	12	8	8	4
Flexible diaphragm (S5a)	NL	NL	NL	12	12	8	8	4
Steel Plate Shear Wall (S6)	NP[c]	NP[c]	NP[c]	NP[c]	NP[c]	NP[c]	NP[c]	NP[c]
Concrete Moment Frames (C1)	NL	NL	NL	12	12	8	8	6
Concrete Shear Walls								
Rigid diaphragm (C2)	NL	NL	NL	12	12	8	8	6
Flexible diaphragm (C2a)	NL	NL	NL	12	12	8	8	6
Concrete Frame with Infill Masonry Shear Walls								
Rigid diaphragm (C3)	NL	NL	NL	12	12	8	8	4
Flexible diaphragm (C3a)	NL	NL	NL	12	12	8	8	4
Precast or Tilt-Up Concrete Shear Walls								
Flexible diaphragm (PC1)	NL	NL	3	2	2	2	2	2
Rigid diaphragm (PC1a)	NL	NL	3	2	2	2	2	2
Precast Concrete Frames								
With shear walls (PC2)	NL	NL	NL	6	6	NP	4	NP
Without shear walls (PC2a)	NL	NL	NL	6	6	NP	4	NP
Reinforced Masonry Bearing Walls								
Flexible diaphragm (RM1)	NL	NL	NL	8	8	8	8	6
Rigid diaphragm (RM2)	NL	NL	NL	8	8	8	8	6
Unreinforced Masonry Bearing Walls								
Flexible diaphragm (URM)	NL	NL	6	4	6	NP	4	NP
Rigid diaphragm (URMa)	NL	NL	6	4	6	NP	4	NP
Seismic Isolation or Passive Dissipation	NP[c]	NP[c]	NP[c]	NP[c]	NP[c]	NP[c]	NP[c]	NP[c]

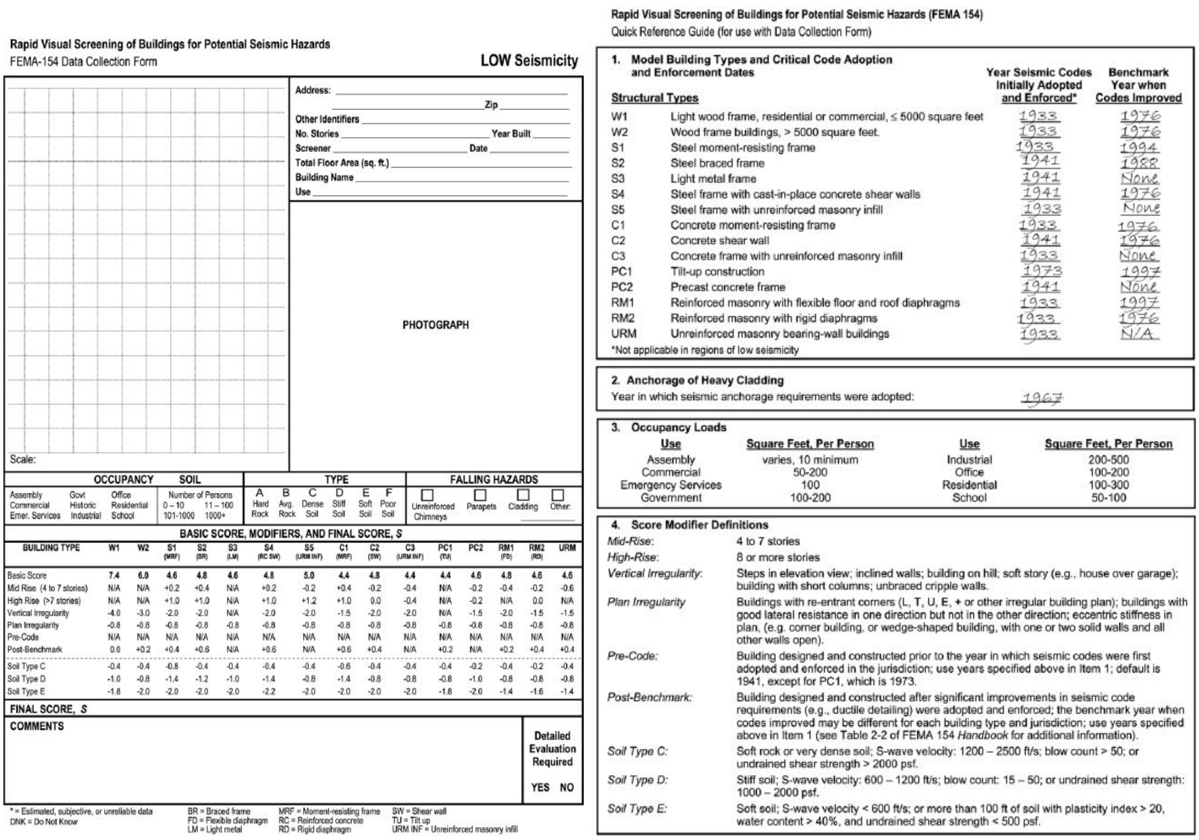
NOTE: The Tier 3 systematic procedures are required for buildings with more than the number of stories listed herein.
[a]Common building types are defined in Section 3.2.1.
[b]Number of stories shall be considered as the number of stories above lowest adjacent grade.
NL = No Limit (No limit on the number of stories).
NP = Not Permitted (Tier 3 systematic procedures are required).
[c]No deficiency-based procedures exist for these building types. If they do not meet the Benchmark Building requirements, Tier 3 systematic procedures are required

Tier 3 Systematic Evaluation, adopting the most robust and comprehensive procedure, can be applied for any structural or non-structural performance levels for every building type. As shown in Figure 3- 10 and Figure 3- 11, the evaluation process can be initiated from Tier 3 without incurring the expense of Tier 1 and 2 if obvious structure deficiencies exist within the structure. However, it is worth noting that the earlier tiers may still be necessary to identify deficiencies other than the obvious ones. If Tier 3 is required for evaluation, it is encouraged to use Tier 1 or 2 evaluation procedures to obtain a general understanding of the building and to find out potential deficiencies before embarking on Tier 3 evaluation. Besides, if Tier 3 is considered as a follow-up-step to Tier 1 and 2 evaluation, the decision of whether to apply Tier 3 evaluation requires judgement concerning the likelihood of Tier 1 and 2 evaluation being too conservative or considering any significant economic or other advantages to perform such a systematic evaluation.

After drawing conclusion from evaluation, engineers may choose either to report deficiencies and recommend mitigation actions, or carry out further evaluation or investigation. If the evaluation is voluntary, the owner may choose to accept the risk of damage from future earthquakes rather than upgrade or demolish the building. If the evaluation is required by a local ordinance for a hazard-reduction program or triggered by a regulation, building code, or policy, the owner may have to choose among retrofit, demolition, occupancy limitations, or other options, as stated in ASCE 41-13.

3.2.2.1. Rapid Visual Screening

Before initiating the three-tiered evaluation process, proper Performance Objective, Building Performance level, Seismic Hazard, and Level of Seismicity should be selected according to ASCE 41-13 Chapter 2, and sufficient as-built information should be collected referring to Chapter 3. While in the absence of building information, a rapid screening process is required, referring to Rapid Visual Screening of Buildings for Potential Seismic Hazards: A Handbook, i.e. FEMA 154 Edition 2. The detailed procedure of Rapid Visual Screening is not discussed in the thesis. Figure 3- 12 shows the data collection form used in Rapid Visual Screening, taking from FEMA 154.



Rapid Visual Screening of Buildings for Potential Seismic Hazards
FEMA-154 Data Collection Form

LOW Seismicity

Address: ______ Zip ______
Other Identifiers ______
No. Stories ______ Year Built ______
Screener ______ Date ______
Total Floor Area (sq. ft.) ______
Building Name ______
Use ______

PHOTOGRAPH

Scale:

OCCUPANCY			SOIL	TYPE						FALLING HAZARDS
Assembly Commercial Emer. Services	Govt Historic Industrial	Office Residential School	Number of Persons 0 – 10 11 – 100 101-1000 1000+	A Hard Rock	B Avg. Rock	C Dense Soil	D Stiff Soil	E Soft Soil	F Poor Soil	☐ Unreinforced Chimneys ☐ Parapets ☐ Cladding ☐ Other:

BASIC SCORE, MODIFIERS, AND FINAL SCORE, S

BUILDING TYPE	W1	W2	S1 (MRF)	S2 (BR)	S3 (LM)	S4 (RC SW)	S5 (URM INF)	C1 (MRF)	C2 (SW)	C3 (URM INF)	PC1 (TU)	PC2	RM1 (FD)	RM2 (RD)	URM
Basic Score	7.4	6.0	4.6	4.8	4.6	4.8	5.0	4.4	4.8	4.4	4.4	4.6	4.8	4.6	4.6
Mid Rise (4 to 7 stories)	N/A	N/A	+0.2	+0.4	N/A	+0.2	-0.2	+0.4	-0.2	-0.4	N/A	-0.2	-0.4	-0.2	-0.6
High Rise (>7 stories)	N/A	N/A	+1.0	+1.0	N/A	+1.0	+1.2	+1.0	0.0	-0.4	N/A	-0.2	N/A	0.0	N/A
Vertical Irregularity	-4.0	-3.0	-2.0	-2.0	N/A	-2.0	-2.0	-1.5	-2.0	-2.0	N/A	-1.5	-2.0	-1.5	-1.5
Plan Irregularity	-0.8	-0.8	-0.8	-0.8	-0.8	-0.8	-0.8	-0.8	-0.8	-0.8	-0.8	-0.8	-0.8	-0.8	-0.8
Pre-Code	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Post-Benchmark	0.0	+0.2	+0.4	+0.6	N/A	+0.6	N/A	+0.6	+0.4	N/A	+0.2	N/A	+0.2	+0.4	+0.4
Soil Type C	-0.4	-0.4	-0.8	-0.4	-0.4	-0.4	-0.4	-0.6	-0.4	-0.4	-0.4	-0.2	-0.4	-0.2	-0.4
Soil Type D	-1.0	-0.8	-1.4	-1.2	-1.0	-1.4	-0.8	-1.4	-0.8	-0.8	-0.8	-1.0	-0.8	-0.8	-0.8
Soil Type E	-1.8	-2.0	-2.0	-2.0	-2.0	-2.2	-2.0	-2.0	-2.0	-2.0	-1.8	-2.0	-1.4	-1.6	-1.4

FINAL SCORE, S

COMMENTS

Detailed Evaluation Required
YES NO

* = Estimated, subjective, or unreliable data
DNK = Do Not Know
BR = Braced frame
FD = Flexible diaphragm
LM = Light metal
MRF = Moment-resisting frame
RC = Reinforced concrete
RD = Rigid diaphragm
SW = Shear wall
TU = Tilt up
URM INF = Unreinforced masonry infill

Rapid Visual Screening of Buildings for Potential Seismic Hazards (FEMA 154)
Quick Reference Guide (for use with Data Collection Form)

1. Model Building Types and Critical Code Adoption and Enforcement Dates

Structural Types		Year Seismic Codes Initially Adopted and Enforced*	Benchmark Year when Codes Improved
W1	Light wood frame, residential or commercial, ≤ 5000 square feet	1933	1976
W2	Wood frame buildings, > 5000 square feet.	1933	1976
S1	Steel moment-resisting frame	1933	1994
S2	Steel braced frame	1941	1988
S3	Light metal frame	1941	None
S4	Steel frame with cast-in-place concrete shear walls	1941	1976
S5	Steel frame with unreinforced masonry infill	1933	None
C1	Concrete moment-resisting frame	1933	1976
C2	Concrete shear wall	1941	1976
C3	Concrete frame with unreinforced masonry infill	1933	None
PC1	Tilt-up construction	1973	1997
PC2	Precast concrete frame	1941	None
RM1	Reinforced masonry with flexible floor and roof diaphragms	1933	1997
RM2	Reinforced masonry with rigid diaphragms	1933	1976
URM	Unreinforced masonry bearing-wall buildings	1933	N/A

*Not applicable in regions of low seismicity

2. Anchorage of Heavy Cladding
Year in which seismic anchorage requirements were adopted: 1967

3. Occupancy Loads

Use	Square Feet, Per Person	Use	Square Feet, Per Person
Assembly	varies, 10 minimum	Industrial	200-500
Commercial	50-200	Office	100-200
Emergency Services	100	Residential	100-300
Government	100-200	School	50-100

4. Score Modifier Definitions

Mid-Rise: 4 to 7 stories
High-Rise: 8 or more stories
Vertical Irregularity: Steps in elevation view; inclined walls; building on hill; soft story (e.g., house over garage); building with short columns; unbraced cripple walls.
Plan Irregularity Buildings with re-entrant corners (L, T, U, E, + or other irregular building plan); buildings with good lateral resistance in one direction but not in the other direction; eccentric stiffness in plan, (e.g. corner building, or wedge-shaped building, with one or two solid walls and all other walls open).
Pre-Code: Building designed and constructed prior to the year in which seismic codes were first adopted and enforced in the jurisdiction; use years specified above in Item 1; default is 1941, except for PC1, which is 1973.
Post-Benchmark: Building designed and constructed after significant improvements in seismic code requirements (e.g., ductile detailing) were adopted and enforced; the benchmark year when codes improved may be different for each building type and jurisdiction; use years specified above in Item 1 (see Table 2-2 of FEMA 154 *Handbook* for additional information).
Soil Type C: Soft rock or very dense soil; S-wave velocity: 1200 – 2500 ft/s; blow count > 50; or undrained shear strength > 2000 psf.
Soil Type D: Stiff soil; S-wave velocity: 600 – 1200 ft/s; blow count: 15 – 50; or undrained shear strength: 1000 – 2000 psf.
Soil Type E: Soft soil; S-wave velocity < 600 ft/s; or more than 100 ft of soil with plasticity index > 20, water content > 40%, and undrained shear strength < 500 psf.

Figure 3- 12: Low seismisity data collection form with a reference guide for a building case showing entries for years in which seismic codes were first adopted and enforced and benchmark years (FEMA 154, and the other two forms-moderate and high seismicity data collection forms are not shown in the thesis)

3.2.2.2. Tier 1 Screening Procedure

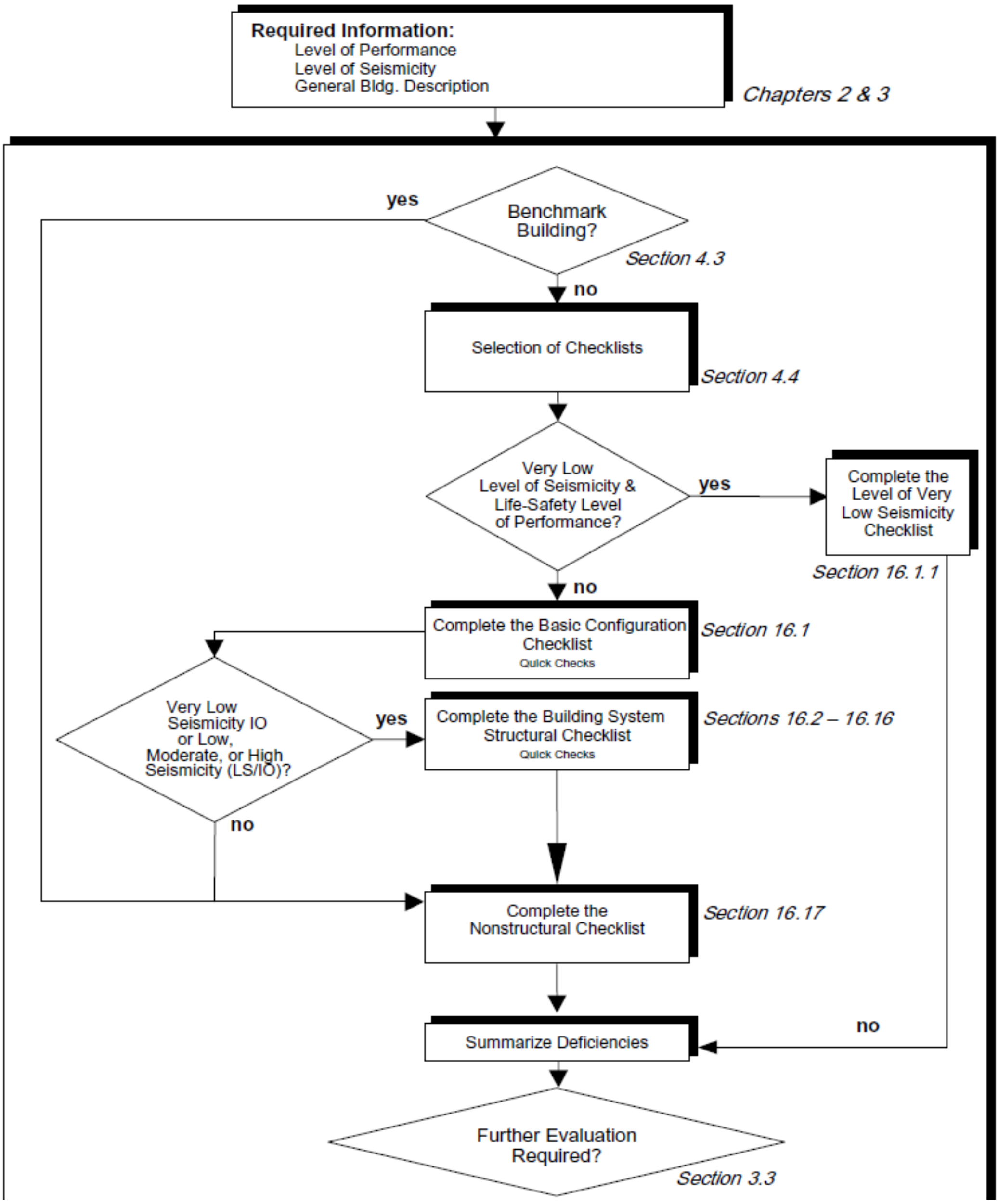


Figure 3- 13: ASCE 41-13 Tier 1 Evaluation Process

By summarising the details presented in ASCE 41-13, and as shown in Figure 3- 13, the inputs required are listed as following:

- Target performance level (S-1, S-3, N-B, N-C)
- Seismic hazard level (BSE1-E for BPOE)
- Level of Seismicity (very low, low, moderate, high)
- On-site investigation and condition assessment (available construction documents, on-site investigation, limited non-destructive investigation)
- Common building type (ASCE 41-13 Table 3-1, 3-2)

- Default material properties (unless otherwise indicated by the available construction documents or by testing) (ASCE 41-13 Table 4-2, 4-3, 4-4, 4-5, Chapter 10)
- Benchmark building information

Table 3- 12: Benchmark Buildings (ASCE41-13 Table 4-6)

Building Type[a,b]	Building Seismic Design Provisions				Seismic Evaluation or Retrofit Provisions		
	NBC[LS] SBC[LS]	UBC[LS]	IBC[LS]	NEHRP[LS]	FEMA 178[LS]	FEMA 310 (1998a)/ ASCE 31[LS,IO]	FEMA 356 (2000)/ ASCE 41[LSc,IOd]
Wood frame, wood shear panels (Types W1 & W2)	1993	1976	2000	1985	[e]	1998	2000
Wood frame, wood shear panels (Type W1a)	[e]	1997	2000	1997	[e]	1998	2000
Steel moment-resisting frame (Types S1 & S1a)	[e]	1994[f]	2000	1997	[e]	1998	2000
Steel concentrically braced frame (Types S2 & S2a)	[e]	1997	2000	[e]	[e]	1998	2000
Steel eccentrically braced frame (Types S2 & S2a)	[e]	1988[f]	2000	1997	[e]	[e]	2000
Buckling-restrained braced frame (Types S2 & S2a)	[e]	[e]	2006	[e]	[e]	[e]	2000
Light metal frame (Type S3)	[e]	[e]	2000	[e]	1992	1998	2000
Steel frame w/ concrete shear walls (Type S4)	1993	1994[7]	2000	1985	[e]	1998	2000
Steel frame with URM infill (Types S5 & S5a)	[e]	[e]	2000	[e]	[e]	1998	2000
Steel plate shear wall (Type S6)	[e]	[e]	2006	[e]	[e]	[e]	2000
Reinforced concrete moment-resisting frame (Type C1)[g]	1993	1994	2000	1997	[e]	1998	2000
Reinforced concrete shear walls (Types C2 & C2a)	1993	1994	2000	1985	[e]	1998	2000
Concrete frame with URM infill (Types C3 & C3a)	[e]	[e]	2000	[e]	[e]	1998	2000
Tilt-up concrete (Types PC1 & PC1a)	[e]	1997	2000	[e]	[e]	1998	2000
Precast concrete frame (Types PC2 & PC2a)	[e]	[e]	2000	[e]	1992	1998	2000
Reinforced masonry (Type RM1)	[e]	1997	2000	[e]	[e]	1998	2000
Reinforced masonry (Type RM2)	1993	1994[7]	2000	1985	[e]	1998	2000
Unreinforced masonry (Type URM)[h]	[e]	1991[i]	2000	[e]	1992	1998	2000
Unreinforced masonry (Type URMa)	[e]	[e]	2000	[e]	[e]	1998	2000
Seismic isolation or passive dissipation	[e]	1991	2000	[e]	[e]	[e]	2000

[a]Building type refers to one of the common building types defined in Table 3-1.
[b]Buildings on hillside sites shall not be considered Benchmark Buildings.
[c]LS: S-3 Structural Performance Level for the BSE-1.
[d]IO: S-1 Structural Performance Level for the BSE-1.
[e]No benchmark year; buildings shall be evaluated using this standard.
[f]Steel moment-resisting frames and eccentrically braced frames with links adjacent to columns shall comply with the 1994 UBC Emergency Provisions, published September/October 1994, or subsequent requirements.
[g]Flat slab concrete moment frames shall not be considered Benchmark Buildings.
[h]URM buildings evaluated or retrofitted and shown to be acceptable using Special Procedure (the ABK Methodology, 1984) may be considered benchmark buildings subject to the limitation of Section 15.2.
[i]Refers to the GSREB or its predecessor, the Uniform Code of Building Conservation (UCBC), or its successor, IEBC Appendix Chapter A1.
[LS]Only buildings designed and constructed or evaluated in accordance with these documents and being evaluated to the Life Safety Performance Level may be considered Benchmark Buildings.
[IO]Buildings designed and constructed or evaluated in accordance with these documents and being evaluated to the Immediate Occupancy Performance Level may be considered Benchmark Buildings.

NBC = *National Building Code*.
SBC = *Standard Building Code*.
UBC = *Uniform Building Code*.
IBC = *International Building Code*.
IEBC = *International Existing Building Code*.
NEHRP = FEMA 368 and 369, *NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings* (BSSC 2000).
FEMA 178.
FEMA 310.
FEMA 356.
ASCE 31-03.
ASCE 41-06.

After obtaining sufficient input data, Benchmark Building criteria (shown in Table 3- 12) should be checked first. If the building meets Benchmark Building criteria, Tier 1 evaluation is no longer required, but a screening of nonstructural components is still needed. If the building is determined as "Noncompliant" with Benchmark Building criteria, Tier 1 evaluation shall be completed together with completion of the checklists specified in ASCE 41-13 Chapter 16. To complete the checklists, simple analyses, together with quick calculations of primary component stiffness or strength, are involved. The detailed calculation procedures are shown in Section 4.2, and the checklists covering rapid evaluation of structural, non-structural, foundation, geologic hazard elements, and site conditions are listed as following, and should be properly selected according to Table 3- 13.

- Very low seismicity checklist
- Basic configuration checklist (performance level (LS, IO) and seismicity (low, moderate, high))
- Building system structural checklist (performance level (LS, IO))
- Nonstructural checklist (performance level (LS, PR))

Table 3- 13: Checklists required for a Tier 1 Screening (ASCE41-13 Table 4-7)

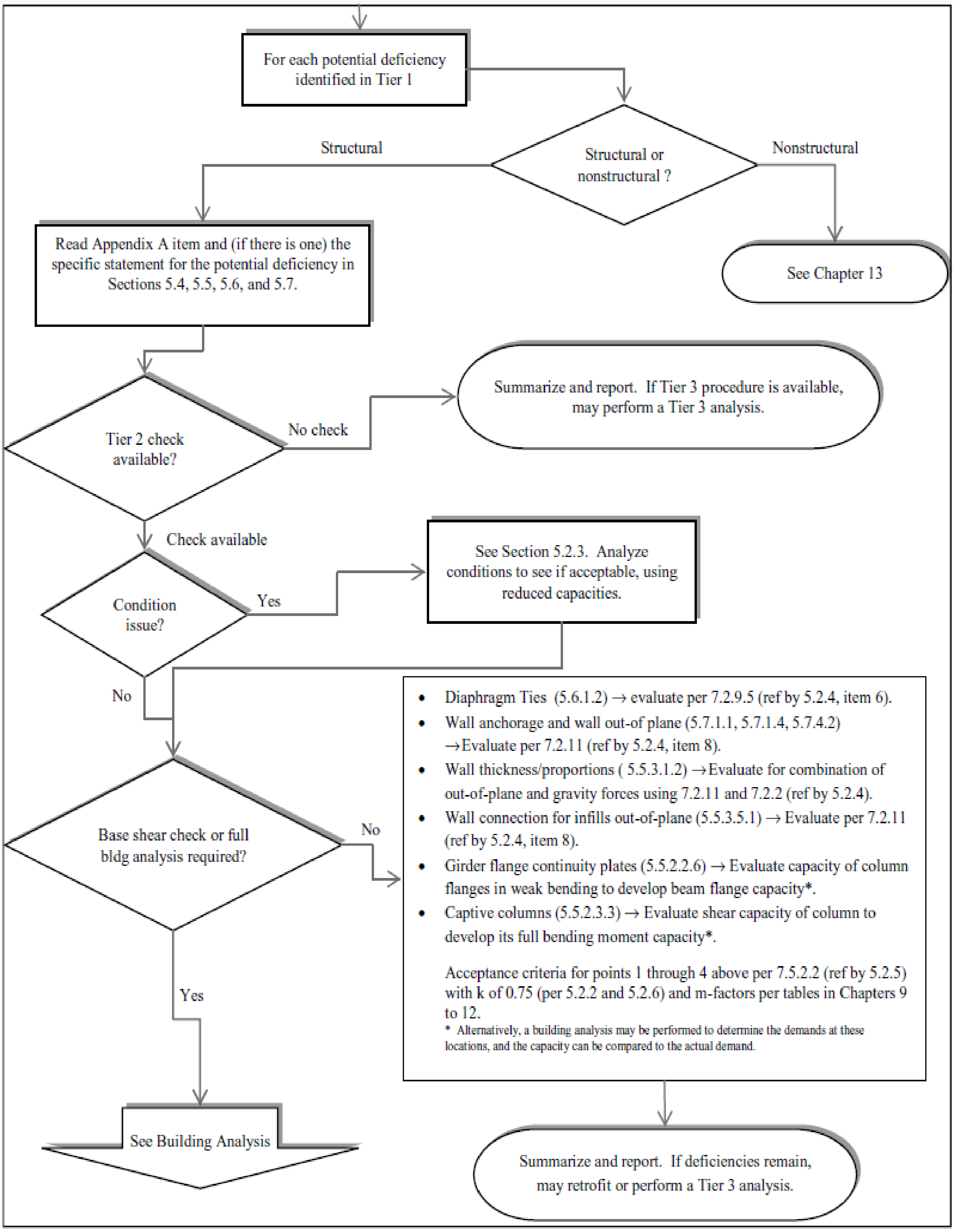
		Required Checklists[a]					
Level of Seismicity[b]	Level of Building Performance[c]	Very Low Seismicity Checklist (Sec 16.1.1)	Basic Configuration Checklist (Sec. 16.1.2)	Life Safety Checklist (Sec. 16.2LS through 16.15LS)	Immediate Occupancy Checklist (Sec. 16.2IO through 16.15IO)	Life Safety Nonstructural Checklist (Sec. 16.17)	Position Retention Nonstructural Checklist (Sec. 16.17)
Very low	LS	X					
Very low	IO		X		X		X
Low	LS		X	X		X	
Low	IO		X		X		X
Moderate	LS		X	X		X	
Moderate	IO		X		X		X
High	LS		X	X		X	
High	IO		X		X		X

[a]An X designates the checklist that must be completed for a Tier 1 screening as a function of the level of seismicity and level of performance.
[b]Defined in Section 2.5.
[c]LS = Life Safety Performance Level, and IO = Immediate Occupancy Performance Level (defined in Section 2.3.3).

The structural deficiencies (i.e. criteria found to be “Noncompliant” in the checklists) that are quickly identified in Tier 1 evaluation may requires further evaluation (i.e. Tier 2 or Tier 3), or engineers may choose to end the investigation then to report the deficiencies and make retrofit suggestions. The limitations of Tier 1 Screening evaluation procedure are highlighted as following:

- Tier 1 is only applicable to the buildings that comply with the common building types defined in ASCE 41-13 Table 3-1 and Table 3-2
- Tier 1 is only applicable to demonstrate compliance with BPOE (Basic Performance Objective for Existing Buildings), and is not permitted to demonstrate compliance with BPON (Basic Performance Objectives Equivalent to New Building Standards). The basic performance objectives for existing buildings are specified in Table 3- 10 (from ASCE41-13 Table 2-1).
- Tier 1 only includes acceptance criteria of structural performance levels S-1, S-3, and non-structural performance levels N-B, N-C.

3.2.2.3. Tier 2 Deficiency-based Evaluation Procedure



(Figure continue in the next page)

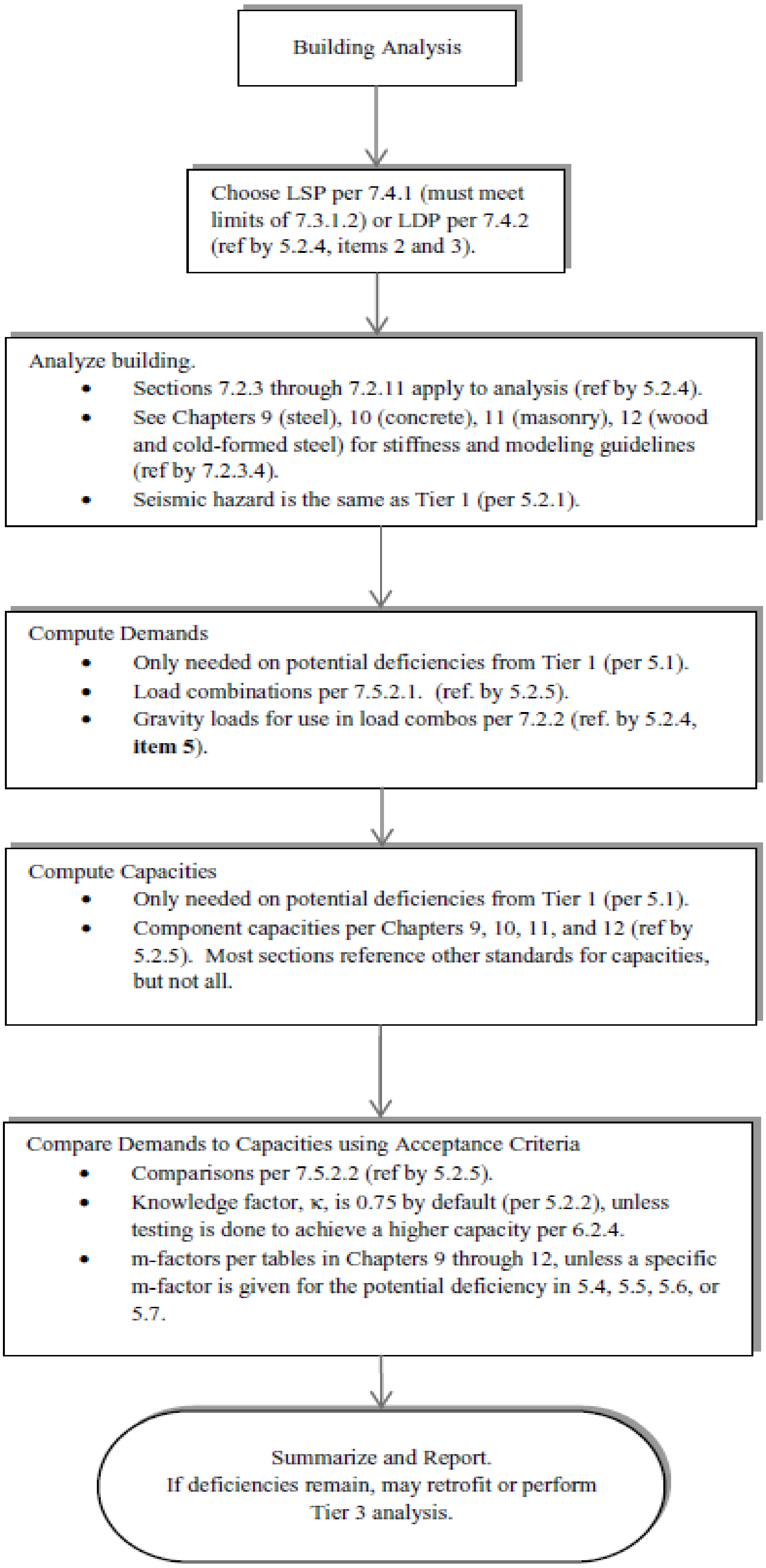


Figure 3- 14: ASCE 41-13 Tier 2 Evaluation Process

In Figure 3- 14, a flowchart of Tier 2 evaluation process is shown, and more details of the procedure are shown in Appendix A4. As shown in the comparison table in Appendix A4, the requirements of input information are quite similar to those defined in Tier 1 evaluation:

- Target performance level (SAME in Tier 1)
- Seismic hazard level (SAME in Tier 1)
- Level of Seismicity (SAME in Tier 1)
- As-built information and condition assessment
  In addition to the requirements specified in Tier 1, destructive examination and testing may be required to perform Tier 2 evaluation. A knowledge factor of 0.75 should be considered during evaluation, unless data collection complies with the requirements for a knowledge factor of 1.0.
- Common building types (SAME in Tier 1)
- Material properties and knowledge factor
  Default material properties or material properties obtained from construction reports or other related documents
- Additional for Linear Dynamic Analysis: response spectrum, site specific response, or ground motion acceleration histories
- Potential deficiencies identified in Tier 1 screening
  It should be noticed that only the deficiencies identified by Tier 1 evaluation are assessed at Tier 2.

Analyses in Tier 2 are limited to linear analysis approaches, Linear Static Procedure (i.e. LSP) and Linear Dynamic Procedure (i.e. LDP), and proper mathematical model should be established, considering 2D or 3D effect (including multidirectional seismic effect), torsion effect, definition of primary and secondary components (including continuity of components, structural sharing common elements), stiffness and strength of components, foundation model with soil-structure interaction defined, damping effect, P-Δ effect, overturning effect, etc. From the analysis, capacities of components associated with the deficiencies identified in Tier 1, and seismic demands can be estimated. Then the determined demands and capacities are used to demonstrate compliance with acceptance criteria (i.e. check that demand not exceeding capacity) for both deformation and force actions. It is worth recognising that Tier 2 evaluation is limited within the scope of the potential deficiencies identified in Tier 1. If the deficiencies are confirmed, engineers may choose to report or carry out Tier 3 evaluation.

The limitations of Tier 2 evaluation are listed as following:

- Tier 2 evaluation only reflects a level of analysis and design that is appropriate for small, relatively simple buildings and the buildings that do not require advanced analytical procedures, and is only applicable to the buildings that comply with the common building categories.

- Tier 2 is only applicable to demonstrate compliance of an existing or retrofit building with BPOE (Basic Performance Objective for Existing Buildings), and is not permitted to demonstrate compliance with BPON (Basic Performance Objectives Equivalent to New Building Standards).
- Tier 2 only includes acceptance criteria of structural performance levels S-1, S-3, and non-structural performance levels N-B, N-C.
- Tier 2 may provide more conservative results compared to Tier 3. A variety of simplifying assumptions are adopted in Tier 2 evaluation.
- Some limitations are associated with analysis approaches, and more detailed information is provided in Chapter 4.

3.2.2.4. Tier 3 Systematic Evaluation

As shown in Appendix A4, the input information required in Tier 3 systematic evaluation includes:

- Target performance level (No constraints)
- Seismic hazard level (No constraints)
- Level of Seismicity
- Data collection and knowledge factor.

  Three knowledge levels, minimum, usual, and comprehensive level, are defined. Corresponding to each knowledge level, the requirements regarding data collection are specified, shown in Table 3- 14 and Table 3- 15. The data may be collected from testing, design drawings or equivalent documents, condition assessment, and so on. Discussions of the application of knowledge levels and factors are shown in Section 3.3.2 and Section 9.2.
- Additional for dynamic analyses: ground motion requirements

Table 3- 14: Data collection requirements corresponding to three levels of knowledge and definition of knowledge factors (Table 6-1 from ASCE 41-13)

Data	Level of Knowledge						
	Minimum			Usual		Comprehensive	
Performance Level	Life Safety or lower			Life Safety or lower		Greater than Life Safety	
Analysis Procedures	LSP, LDP			All		All	
Testing	No tests			Usual testing		Comprehensive testing	
Drawings	Design drawings or equivalent		No drawings	Design drawings or equivalent		Construction documents or equivalent	
Condition assessment	Visual	Visual	Comprehensive	Visual	Comprehensive	Visual	Comprehensive
Material properties	From default values	From design drawings	From default values	From drawings and tests	From usual tests	From documents and tests	From comprehensive tests
Knowledge factor (κ)	0.75	$0.9^{a,b}$	0.75	1.00	1.00	1.00	1.00

NOTE: LSP, linear static procedure; LDP, linear dynamic procedure.
[a]If the building meets the benchmark requirements of Table 4-5, then $\kappa = 1.0$.
[b]If inspection or testing records are available to substantiate the design drawings, then $\kappa = 1.0$.

Table 3- 15: Data collection requirements for the three knowledge levels

Minimum data collection requirements	Usual data collection requirements	Comprehensive data collection requirements
• Design drawings shall show, at minimum, the configuration of the gravity load system and seismic-force-resisting system and typical connections with sufficient detail to carry out linear procedures (where design drawings are available, information shall be verified by a visual condition assessment) • In the absence of sufficient information from design drawings, incomplete or non-existent information shall be supplemented by a comprehensive condition assessment, including destructive and non-destructive investigation • In the absence of material test records and quality assurance reports, use of default material properties • Information needed on adjacent buildings shall be gained through field surveys and research of as-built information made available by the owner of the subject building • Site and foundation information shall be collected	• Design drawings shall show, as a minimum, the configuration of the gravity load system and seismic-force-resisting system and typical connection with sufficient detail to carry out the selected analysis procedure (where design drawings are available, information shall be verified by a visual condition assessment) • In the absence of sufficient information from design drawings, incomplete or non-existent information shall be supplemented by a comprehensive condition assessment, including destructive and non-destructive investigation • In the absence of material test records and quality assurance reports, material properties shall be determined by usual materials testing • Information needed on adjacent buildings shall be gained through field surveys and research of as-built information made available by the owner of the subject building • Site and foundation information shall be collected	• Information shall be obtained from construction documents including design drawings, specifications, material test records, and quality assurance reports covering original construction and subsequent modifications to the structure (where construction documents are available, information shall be verified by a visual condition assessment) • If construction documents are incomplete, missing information shall be supplemented by a comprehensive condition assessment, including destructive and non-destructive investigation • In the absence of material test records and quality assurance reports, material properties shall be determined by comprehensive materials testing, including the limitations on the coefficient of variation • Information needed on adjacent buildings shall be gained through field surveys and research of as-built information made available by the owner of the subject building • Site and foundation information shall be collected

In Tier 3 evaluation, four analyses (Linear Static Procedure, Linear Dynamic Procedure, Nonlinear Static Procedure, and Nonlinear Dynamic Procedure) can be selected. Nonlinear analysis tools, such as SAP 2000 Advanced, STAAD Pro Nonlinear, PERFORM, ANSYS, etc., can be used for modelling. In some of these analysis tools, the component modelling parameters and numerical acceptance criteria corresponding to different limit states that are specified in ASCE 41-13 are implemented. Therefore, response of both the global structure and individual components can be obtained directly, and potential deficiencies can then be identified. Discussions regarding the modelling parameters and numerical acceptance criteria are in Section 3.3.4. The limitations of Tier 3 systematic evaluation are listed as following:

- Performing Tier 3 evaluation can be complex, expensive and time-consuming, in spite of the fact that more accurate results can be computed. However, it is worth noting that Tie 3 may still result in construction savings equal to many times of their cost under some circumstances, for instance, for a very complex structure that is under consideration, or a very importance building with obvious structural deficiencies.
- Tier 3 evaluation is not applicable when $\mu_{strength}>\mu_{max}$. (Details are shown in Appendix A4)
- Some limitations are associated with analysis approaches, and more detailed information is provided in Chapter 4.

3.2.3. EN 1998-3: 2005 (European Code) and NTC 2008 (Italian Code)

Figure 3- 15 and Figure 3- 16 illustrate the development of European and Italian code provisions of building assessment. It is found that EN 1998-3: 2005 and NTC 2008 adopt similar assessment procedures, and the main differences lie in analysis approaches that are applied. Detailed discussions regarding the analyses are shown in Chapter 4.

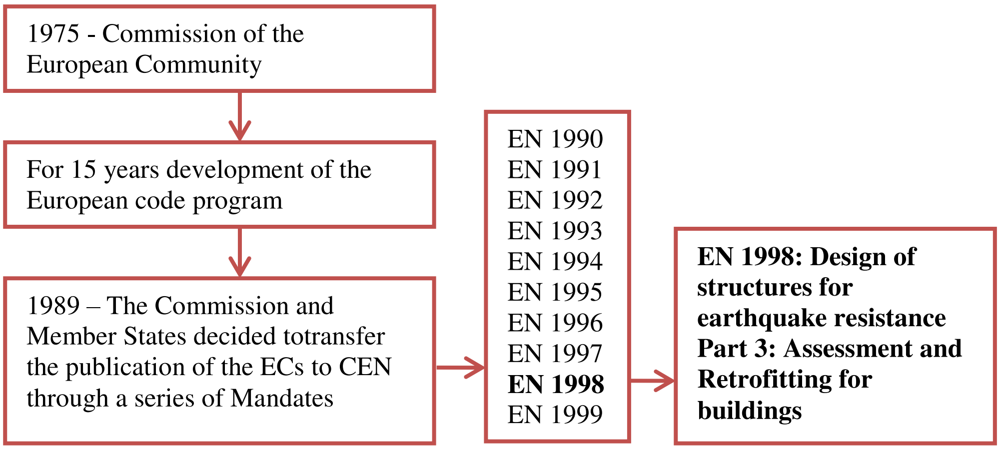


Figure 3- 15: Timeline of development of European Code (A, De Pra and S, Bianchi)

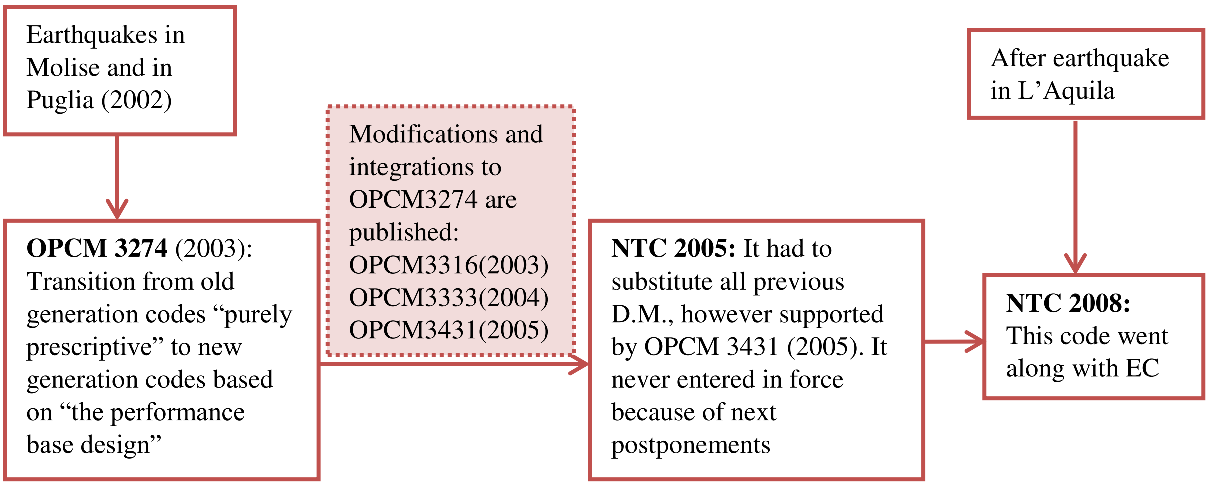


Figure 3- 16: Timeline of development of Italian Code (A, De Pra and S, Bianchi)

3.2.3.1. Evaluation of Knowledge Level

In both European and Italian code provisions, an initial evaluation on structure to determine knowledge level (KL) is specified. From a variety of sources, such as design drawings, field investigations, laboratory tests, etc., the input data shall be collected, according to the listed requirements showing as follows.

- Identification of structural system (compliance with regularity criteria)
- Identification of type of building foundations
- Identification of ground conditions

- Information concerning component geometry and properties
- Information concerning material mechanical properties
- Information concerning material defects and inadequate detailing
- Information concerning the seismic design criteria that were applied
- Description of use of the buildings
- Re-assessment of imposed actions
- Information concerning type of structural damage

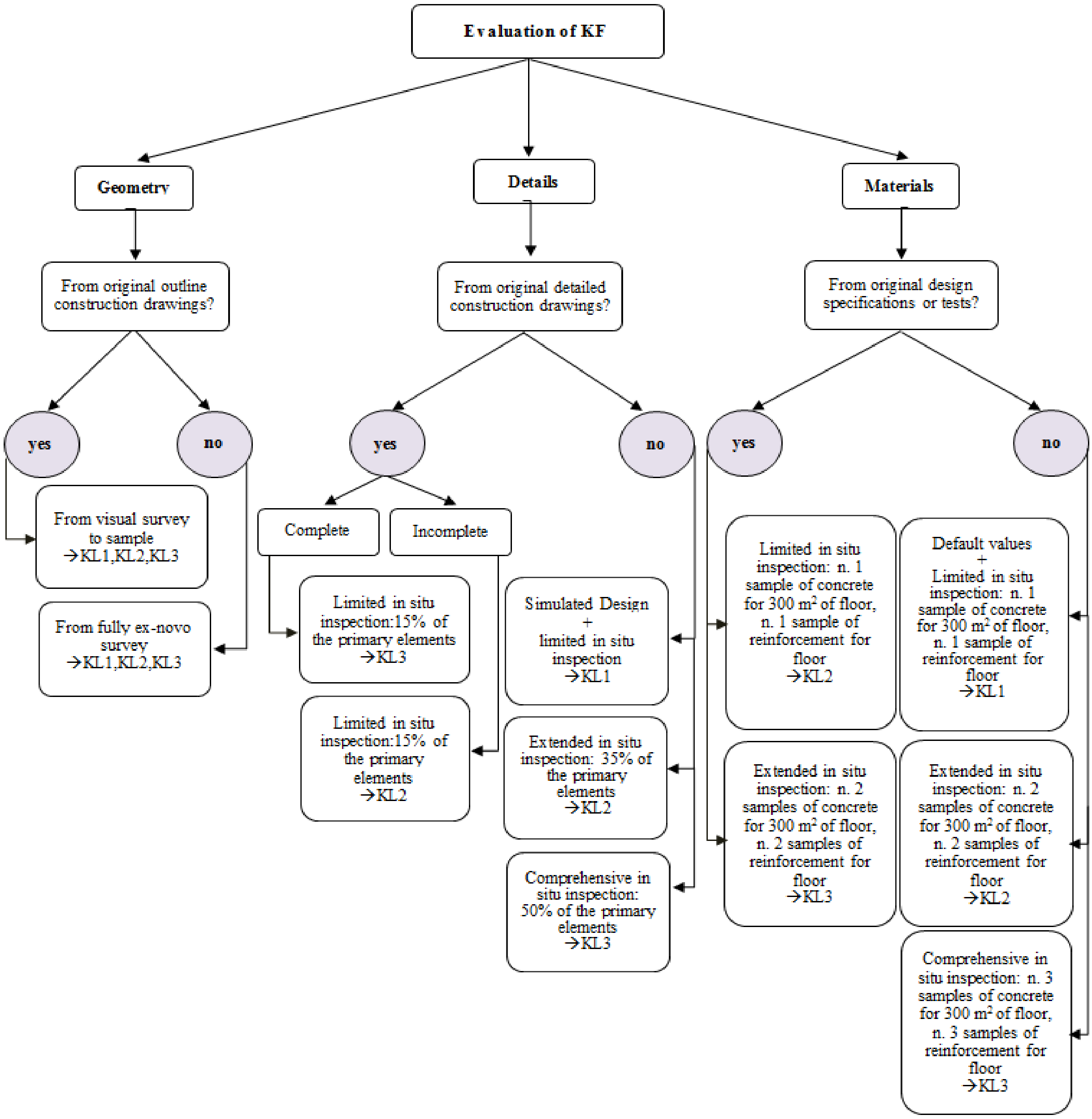


Figure 3- 17: Evaluation of knowledge factor process

The collected data can be categorised as "geometry, detailing and material". The "geometry category" includes geometrical properties of structural components and nonstructural elements; the "detailing category" consists of amount and detailing of the reinforcements applied; and the "material category" consists of mechanical properties of the constituent materials. Depending on the quantity and quality of the collected data, appropriate knowledge level (i.e. KL1, KL2, and KL3) can be determined. With the determined knowledge level, the type of analysis and the value of the confidence factors can thus be decided, according to Figure 3- 17.

It is worth recognising that unlike NZSEE 2006 and ASCE 41-13, neither the European nor the Italian code provisions include an initial assessment of the structure to give a preliminary determination of potential structural deficiencies. As discussed in previous, in EN 1998:2005 and NTC 2008, the first stage of the assessment procedures only has the scope to define knowledge level. It has also been found that the determination of knowledge factor is dependent on the predictive global response of the structure, which is inappropriate. Discussions regarding this issue are shown in Section 3.3.2.

3.2.3.2. Knowledge Level 1

As specified in EN 1998:2005 and NTC 2008, knowledge level 1 is associated with a more comprehensive evaluation process compared to preliminary screening process. With the input data collected (i.e. three categories – geometry, detailing and material, together with ground and earthquake conditions), two analysis approaches are suggested, shown in the following.

- Lateral Force Procedure (i.e. LSP).
- Modal Response Spectrum Analysis (i.e. LDP)

It is worth noting that a reduction factor q should be applied to approximate the nonlinear response of the structure in linear analyses. The calculated actions or capacities should be checked with acceptance criteria that are defined in terms of chord rotation and shear under various limit states. Detailed information concerning the simplified formulae to estimate fundamental period, the calculation of base shear force, the approximation of structural mass and effective height, the number of modes required to conduct modal analysis, the appropriate combination rules, the limitations associated with the analyses, etc., is presented in Chapter 4.

3.2.3.3. Knowledge Level 2 and Knowledge Level 3

For knowledge level 2 and 3, comprehensive and systematic seismic assessment procedures are suggested in EN 1998:2005 and NTC 2008. It is worth noting that knowledge level 3 requires more efforts during collecting input data. At both levels, same analysis approaches are suggested:

- Lateral Force Procedure (i.e. LSP)
- Modal Response Spectrum Analysis (i.e. LDP)
- Nonlinear Static Analysis (i.e. NSP)
- Nonlinear History Analysis (i.e. NDP)

Detailed information regarding the analyses is shown in Chapter 4.

3.3. Critical Comparison among Different Codified Procedures

In this section, conclusions drawn from critical comparisons among NZSEE 2006, ASCE 41, EC8 (or NTC2008) are presented. The conclusions are focused on five parts shown as follows, and sections 3.3.1 to 3.3.5 provide concise discussions regarding each of these aspects. The detailed comparison table is provided in Appendix A4.

- Adoption of preliminary evaluation procedure
- Application of knowledge level and the corresponding knowledge factors (confidence factors)
- Assessment at material level,
- Assessment at component level
- Differences in analysis approaches adopted in the four codified assessment procedures

3.3.1. Preliminary Evaluation Procedure

Table 3- 16: Preliminary evaluation procedures specified in the four codified assessment procedures

<table>
<tr><th colspan="2"></th><th colspan="4">Codified Seismic Assessment/Evaluation Procedures</th></tr>
<tr><th colspan="2">Level/Stage of Assessment/ Evaluation</th><th>NZSEE 2006</th><th>ASCE 41-13</th><th>EN 1998-3: 2005</th><th>NTC 2008</th></tr>
<tr><td rowspan="7">Screening with Minimal Level of Analysis Applied</td><td>Specific Title in Each Code/Guideline</td><td>Initial Seismic Assessment (ISA)</td><td>Tier 1 Screening Procedure</td><td>Evaluation of Knowledge Level</td><td>Evaluation of Knowledge Level</td></tr>
<tr><td>Inputs and Requirements</td><td>Age, importance level, construction type, previous strengthening, location, soil type, building height</td><td>Target performance level, seismic hazard level, level of seismicity, on-site investigation and condition assessment, common building type, default material properties, benchmark building information</td><td>Drawings and equivalent information, on-site investigation, contemporary standard, in-situ and/or laboratory measurements and tests</td><td>Structural and geotechnical reports, executive details, in situ and/or laboratory measurements and tests</td></tr>
<tr><td>Analysis Approach</td><td>None</td><td>Simplified LSP</td><td>None</td><td>None</td></tr>
<tr><td>Evaluation Procedures and Applied Tools</td><td>Preliminary Screening; Prioritisation; IEP</td><td>Quick checks for strength and stiffness; Checklists</td><td>Initial screening</td><td>Initial screening</td></tr>
<tr><td>Acceptance Criteria</td><td>CSWs (severe, significant and insignificant), %NBS, EPB and ERB</td><td>Detailed acceptance criteria are stated in the checklists</td><td>None</td><td>None</td></tr>
<tr><td>Outputs</td><td>CSWs ,%NBS, NZSEE Grades</td><td>Completion of checklists with potential deficiencies identified.</td><td>Knowledge level and corresponding analysis method and determination of parameters</td><td>Knowledge level and corresponding analysis method and determination of parameters</td></tr>
<tr><td>Limitations</td><td>Results not very reliable due to many reasons; ULS only; not considering many factors, like nonstructural elements, adjacent buildings</td><td>Only for common building types; only to demonstrate compliance with BPOE; only include acceptance criteria for Immediate Occupancy and Life Safety for structural performance level, and Position Retention and Life Safety for nonstructural</td><td>The confidence factor is determined based on general characteristics, but is applied at “Material Level”, which is inappropriate.</td><td>SAME in EN 1998-3 : 2005</td></tr>
</table>

As shown in Table 3- 16, different preliminary evaluation procedures are specified in the four assessment guidelines or standards. In NZSEE 2006 and ASCE 41-13, comparatively regulated procedures of initial evaluation of the structure are provided. However, it has been found that in EN1998-3: 2005 and NTC 2008, even though a screening process is required to check the structure before carrying out any further levels of evaluation, there are no established procedures. Instead of giving detailed preliminary evaluation procedures, the first step of evaluation process suggested by EN1998-3: 2005 and NTC 2008 is to determine knowledge levels based on the quality of input data, and then to select an appropriate analysis method. It is worth noting that in ASCE 41-13, at this stage, does not involve defining different knowledge levels. The following list provides a summary of the differences found between the preliminary evaluation procedures from NZSEE 2006 and ASCE 41-13, and more detailed information is shown in Appendix A4.

- Compared to NZSEE 2006 ISA procedure, ASCE41-13 Tier 1 evaluation procedure is more component-based, i.e. Tier 1 evaluation procedure is associated with the evaluation of components, such as simple calculation of component capacities, completion of the checklists associated with primary, secondary, and nonstructural components. In spite of the fact that NZSEE 2006 ISA procedure does provide quick checks of some particular components that are more prone to structural deficiencies, for examples, short columns, transferring components like cantilever beams, etc., the procedure still mainly focuses on the evaluation of the entire structure and tends to identify potential deficiencies at global level.
- ASCE 41-13 contains comprehensive information regarding Benchmark Building Criteria, which may lead to a significant saving in time and expense by avoiding further levels of investigation. While NZSEE 2006 includes very brief description of the building types that are not potentially EPBs which thus may not require further level of evaluation. Therefore, it can be suggested that more assessment instructions associated with the structures conforming to the design standards when they were built, should be provided.
- ASCE 41-13 provides checks of nonstructural components in all tiers, with an increase of sophistication level from the lower to the upper tier. However, no assessment procedures of nonstructural components are found in NZSEE 2006.
- By performing NZSEE 2006 ISA and ASCE 41-13 Tier 1 evaluation, potential deficiencies (i.e. critical structural weaknesses) can be identified. Other assessment outputs include %NBS from NZSEE 2006 and checklists (of entire structure and components) from ASCE 41-13.

For typical building cases, preliminary assessment is necessary. The advantages of carrying out preliminary assessment are listed as following:

- Help engineers to get familiar with buildings, structural systems, material properties, and other information

- Identify potential deficiencies without incurring the expense of further level of evaluation, thereby saving time and expense

In aiming to improve NZSEE 2006 ISA procedure, the followings may be suggested:

- More instructions regarding assessment at component level need to be provided, including both structural and nonstructural components, e.g. to provide checklists similar to those in ASCE 41-13.
- More instructions may be required to define non-EPBs without proceeding to further level of assessment for the buildings conforming to the design standards.

3.3.2. Knowledge Level and Knowledge Factor (or Confidence Factor)

Table 3- 17: Knowledge levels and knowledge factors applied in the four codified assessment procedures

		Codified Seismic Assessment/Evaluation Procedures			
Level/Stage of Assessment/ Evaluation		**NZSEE 2006**	**ASCE 41-13**	**EN 1998-3: 2005**	**NTC 2008**
Screening with Minimal Level of Analysis Applied	Specific Title in Each Code/Guideline	Initial Seismic Assessment (ISA)	Tier 1 Screening Procedure	Evaluation of Knowledge Level	Evaluation of Knowledge Level
	Inputs and Requirements				
	Analysis Approach				
	Evaluation Procedures and Applied Tools				
	Acceptance Criteria				
	Outputs				
	Limitations				
More Detailed Assessment than Screening	Specific Title in Each Code/Guideline	NONE	Tier 2 Deficiency-based Evaluation	**Knowledge Level 1 Confidence Factor**	**Knowledge Level 1 Confidence Factor**
	Inputs and Requirements		**Knowledge Factor**		
	Analysis Approach		LSP LDP	LSP LDP	LSP LDP
	Evaluation Procedures and Applied Tools				
	Acceptance Criteria		**κ**	**CF**	**CF**
	Outputs				
	Limitations				
Detailed Assessment	Specific Title in Each Code/Guideline	Detailed Seismic Assessment (DSA)	Tier 3 Systematic Evaluation	**Knowledge Level 2 Knowledge Level 3 Confidence Factor**	**Knowledge Level 2 Knowledge Level 3 Confidence Factor**
	Inputs and Requirements		**Knowledge Level Knowledge Factor**		
	Analysis Approach	LSP LDP SLaMa NSP NDP	LSP LDP NSP NDP	LSP LDP NSP NDP	LSP LDP NSP NDP
	Evaluation Procedures and Applied Tools	General FB DB			
	Acceptance Criteria		LSP/LDP **κ** NSP/NDP **κ**	**CF**	**CF**
	Outputs	LSP LDP SLaMa NSP NDP	LSP/LDP NSP NDP		
	Limitations				

Table 3- 17 demonstrates the application of knowledge levels and knowledge factors (or confidence factors) in the four codified assessment procedures.

NZSEE 2006 Section 4.7 (d) states that "ATC 33.03 (ATC 1995) establishes three categories of building information, corresponding to good, fair and poor information classes. Reference to ATC 33.03 may be of assistance in determining what, if any, allowance to make". However, there is no specific qualitative or quantitative definition of knowledge levels or knowledge factors, though it is obvious that the two stages of assessment process – ISA and DSA – require different levels of input data along with assessment procedures and analyses of different complexity. Unlike NZSEE 2006, both ASCE 41-13 and EN 1998-3:2005 (NTC 2008 is similar to EN 1998-3:2005) have clear definition of knowledge levels, and the corresponding knowledge factors (or confidence factors) are applied the assessment.

However, the difference between ASCE 41-13 and EN 1998-3:2005 regarding the determination of knowledge factors (or confidence factors) should be highlighted. In ASCE-41-13, three knowledge levels – minimum, usual, or comprehensive – are specified. The determination of knowledge level should take account for objective of performance level, availability of analysis tools, quality and quantity of input information, etc., as tabulated in Table 3- 18. Table 3- 18 also provides suggested values of knowledge factor (**κ**) which accounts for any uncertainty associated with component as-built information. The values of the factor are established based on the access to original construction documents or condition assessments, destructive or non-destructive testing of the representative components (i.e. before assessment at material level), and should be used in the evaluation of component capacities (i.e. at component level), as shown in Table 3- 19. In EN 1998-3:2005 and NTC 2008, three knowledge levels are also defined, as shown in Table 3- 20. Similar to ASCE 41-13 procedure, confidence factors are applied in order to account for the uncertainty associated with component as-built information. However, unlike specified in ASCE 41-13, the values of confidence factor are determined based on the characteristics and the expected global mechanism of the structure, and the factors are applied to material properties (i.e. at material level), which is inappropriate and needs to be modified.

In ASCE 41-13, it is also stated that the extent of testing or the use of knowledge factors is permitted to be waived if it is determined that at the time of construction there were adequate testing or inspection processes in place to justify the properties specified in the design drawings. In other words, the knowledge factors are not required if there is good confidence in the material or component information (i.e. κ=1).

Table 3- 18: Data collection requirements in Tier 3 Evaluation, three knowledge levels(ASCE 41-13Table 6-1)

Data	Level of Knowledge: Minimum			Usual		Comprehensive	
Performance Level	Life Safety or lower			Life Safety or lower		Greater than Life Safety	
Analysis Procedures	LSP, LDP			All		All	
Testing	No tests			Usual testing		Comprehensive testing	
Drawings	Design drawings or equivalent		No drawings	Design drawings or equivalent		Construction documents or equivalent	
Condition assessment	Visual	Visual	Comprehensive	Visual	Comprehensive	Visual	Comprehensive
Material properties	From default values	From design drawings	From default values	From drawings and tests	From usual tests	From documents and tests	From comprehensive tests
Knowledge factor (κ)	0.75	$0.9^{a,b}$	0.75	1.00	1.00	1.00	1.00

NOTE: LSP, linear static procedure; LDP, linear dynamic procedure.
[a]If the building meets the benchmark requirements of Table 4-5, then $\kappa = 1.0$.
[b]If inspection or testing records are available to substantiate the design drawings, then $\kappa = 1.0$.

Table 3- 19: Calculation of component action capacity in linear (left)and nonlinear analyses (right) (ASCE 41-13, Table 7-6 and 7-7)

Parameter	Deformation Controlled	Force Controlled
Existing material strength	Expected mean value with allowance for strain-hardening	Lower-bound value (approximately mean value 1σ level)
Existing action capacity	$\kappa\, Q_{CE}$	$\kappa\, Q_{CL}$
New material strength	Expected material strength	Specified material strength
New action capacity	Q_{CE}	Q_{CL}

Parameter	Deformation Controlled	Force Controlled
Deformation capacity (existing component)	$\kappa \times$ Deformation limit	N/A
Deformation capacity (new component)	Deformation limit	N/A
Strength capacity (existing component)	N/A	$\kappa \times Q_{CL}$
Strength capacity (new component)	N/A	Q_{CL}

$$kQ_{CL} > Q_{UF},\ Q_{UF} = Q_G \pm \frac{Q_E}{C_1 C_1 J}\ \text{(forcecontrolledactions)},\ mkQ_{CE} > Q_{UD},\ Q_{UD} = Q_G + Q_E\ \text{(deformation − controlled actions)}$$

Table 3- 20: Knowledge levels and corresponding analysis approaches (EN 1998-3: 2005, Table 3.1)

Knowledge Level	Geometry	Details	Materials	Analysis	CF
KL1	From original outline construction drawings with sample **visual** survey ***or*** from **full** survey	Simulated design in accordance with relevant practice ***and*** from **limited** *in-situ* inspection	Default values in accordance with standards of the time of construction ***and*** from **limited** *in-situ* testing	LF- MRS	CF_{KL1}
KL2		From incomplete original detailed construction drawings with **limited** *in-situ* inspection ***or*** from **extended** *in-situ* inspection	From original design specifications with **limited** *in-situ* testing ***or*** from **extended** *in-situ* testing	All	CF_{KL2}
KL3		From original detailed construction drawings with **limited** *in-situ* inspection ***or*** from **comprehensive** *in-situ* inspection	From original test reports with **limited** *in-situ* testing ***or*** from **comprehensive** *in-situ* testing	All	CF_{KL3}

NOTE The values ascribed to the confidence factors to be used in a country may be found in its National Annex. The recommended values are $CF_{KL1} = 1,35$, $CF_{KL2} = 1,20$ and $CF_{KL3} = 1,00$.

LF – Lateral Force procedure; MRS – Modal Response Spectrum analysis; CF – Confidence Factors

Based on past experience and studies of alternative assessment procedures around the world, it is reasonable to suggest that qualitative definitions of knowledge levels should be adopted in New Zealand assessment guidelines, which can be similar to the specifications provided in Table 6-1 from ASCE 41-13. However, the application of knowledge factors (or confidence factors) may need future investigation. More discussions regarding this issue are presented in Chapter 5 and 9.

3.3.3. Assessment at Material Level

Information associated with material properties is usually sourced from construction documents, on site surveys or investigations, and physical testing of material. Table 3- 21 lists the sources of material properties or strengths specified in the four codified assessment procedures. Apart from construction documents, on site investigations and physical testing, ASCE 41-13 also suggests that the information may be gathered by interviewing building owners, tenants, managers, the original architects and engineers, contractors and the local building officials. In addition, EN 1998-3: 2005 recommends that if different sources are available, then cross-checks should be carried out between the data collected from the different sources, in order to minimise uncertainties in the material information.

Table 3- 21: Sources of material properties specified in the four codified assessment procedures

Material	NZSEE 2006	ASCE 41-13	EN 1998-3: 2005	NTC 2008
Source of Information	Original construction documents, or surveys, or physical testing of representative samples of materials.	Available construction documents and other records, e.g. building design data, including contract drawings, specifications, and calculations, maintenance histories, manufacturers' literature and test data, reference standards and codes from the period of construction, other data such as assessments of the building performance during past earthquakes. On site investigation (field verification) and condition assessment - To verify conformance of conditions described in available documents - To identify alternations or deviations - To supplement incomplete documents - To confirm quality of construction and maintenance (e.g. no significant deterioration of structural materials) - Then select and review the appropriate sets of evaluation statements Destructive and non-destructive examination and testing of selected building materials. Interviews with building owners, tenants, managers, the original architect and engineer, contractors and the local building official.	Available documentation specific to the building in question Relevant generic data sources(contemporary codes and standards) Field investigations In-situ and/or laboratory measurements and tests Cross-checks should be made between the data collected from different sources to minimise the uncertainties	Design documentation related to the building Acquired documentation after the time of construction Visual survey In-situ or laboratory tests

As shown in Table 3- 22, a summary of general requirements of material properties and strengths specified in the four codified assessment procedures is provided. NZSEE 2006 Guidelines, as discussed in previous, unlike the other three assessment provisions, do not include clear specifications regarding knowledge levels, and the guidelines suggest that probable material strength obtained from either design and construction documents or physical test results should be used. ASCE 41-13, EN 1998-3:2005 and NTC 2008, with clearly definition of knowledge levels together with the application of knowledge factors (or confidence factors), suggest that either default values based on the available construction documents or physical testing results should be applied. Under the circumstance where only limited information is available, material properties and strengths can be approximated based on material history used in design and construction in that country. It is found that only ASCE 41-13 provides completed and detailed material history tables, as shown in Section 3.3.3.1 and Section 3.3.3.2. NZSEE 2006 only notifies a few references of material properties applied in old days, for instance, old concrete and reinforcing steel pre 1970s, which may not be sufficient for assessment. NZSEE 2006 also specifies varying ranges of material properties in the absence of data; however, the reliability of these ranges should be reviewed.

Table 3- 22: General requirements of material properties

Material	NZSEE 2006	ASCE 41-13	EN 1998-3: 2005	NTC 2008
General	If in an absence of data, variations considered should be one standard deviation or±20% of the mean. Original design values are NOT appropriate. Probable strength (expected strength) should be determined from available information, e.g. default values (NOTE: reliability of information), or should be based on actual test results, e.g. actual mean of tests (NOTE: effect of variations from the mean) NOTE: Building material may vary from member to member, and effect should be taken into account. ATC 33.03 (ATC 1995) establishes three categories of building information, corresponding to good, fair and poor information classes.	Concrete compressive strength, yield and ultimate strength of conventional and pre-stressing reinforcing steel and metal connection hardware. Other: tensile strength and modulus of elasticity of concrete, ductility, toughness, and fatigue properties of concrete, carbon equivalent present in the reinforcing steel, presence of any degradation such as corrosion or deterioration of bond between concrete and reinforcement. Apply default values (Tables in Chapter 7 to 10) are to be assumed: (conservative/ lower bound). It should be noted that this standard does not permit the use of default material properties for Tier 2 and Tier 3 evaluations without the application of the knowledge factor. Unless otherwise indicated by the available construction documents (conservative/ lower bound) Or by testing: (material testing is required to achieve a knowledge factor of 1.0) - Limited non-destructive investigation s for Tier 1 (where required) - Destructive examination and testing may be required for Tier 2 (more information required) - Non-destructive examination and testing required for Tier 3 - Test methods and minimum number of test are specified in 10.2.2.3 and 10.2.2. Knowledge level (minimum, usual, comprehensive) for Tier 2 and 3	From available information: default values in accordance with the standard of the time of construction or values from original design specifications Based on test results: Concrete: compressive strength, uniformity, quality, presence and location of internal damage, density and thickness of internal damage and voids Steel: yield strength, tensile strength, hardness, deterioration and potential corrosion Depending on the level of inspection (limited, extended and comprehensive), the minimum number of tests on materials is defined in EN 1998-3:2005 Table 3.2. Material information, in addition to geometry and details, define three knowledge levels. For each KL is associated a confidence factor (CF), that is applied to material strengths to take into account the uncertainties. (EN 1998-3:2005 Table3.1)	From available information: default values in accordance with the standard of the time of construction or values from original design specifications or original test reports Based on test results: Concrete: compressive strength, uniformity, quality, presence and location of internal damage, density and thickness of internal damage and voids Steel: yield strength, tensile strength, hardness, deterioration and potential corrosion Three levels of inspection are defined (limited, extended and comprehensive) and the minimum number of tests (similar to EN 1998-3:2005 Table 3.2). According to the amount and quality of information is possible to define the KL and then the CF (similar toTable3.1). The CF will be applied to material strength for the calculation of component capacities, in addition to partial safety factors

3.3.3.1. Concrete

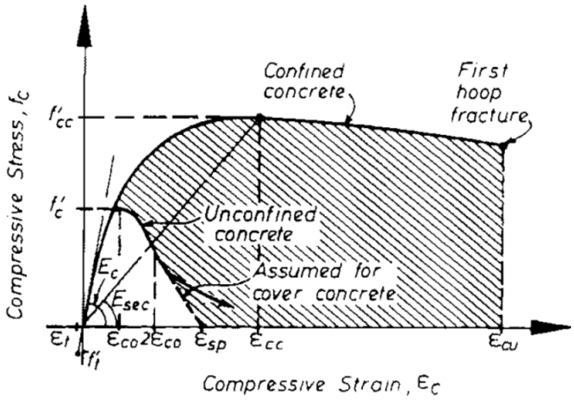


Figure 3- 18: Stress-strain model for monotonic loading of confined and unconfined concrete in compression (Paulay, T., Priestley, M.J.N., Seismic Design of Reinforced Concrete and Masonry Buildings)

Concrete properties and strengths considered in the assessment of reinforced concrete structures are listed in the following, and are also shown in Figure 3- 18.

- Compressive stress (fc') and ultimate strain (εc) for unconfined concrete
- Compressive stress (fcc') and concrete strain (εcc, εcu) for confined concrete

  In order to obtain the confined concrete properties, reinforcing details (incl. longitudinal reinforcement, transverse reinforcement, confinement, additional shear reinforcement) are required.

In Table 3- 23, the procedures to determine of concrete properties and strengths specified in the four codified assessment procedures are shown. In all the four codified assessment procedures, sufficient guidelines regarding the determination of concrete compressive strength are provided; however, only NZSEE 2006 and ASCE 41-13 include guidelines for the determination of concrete strain properties.

According to NZSEE 2006 Guidelines, probable material strength (i.e. mean strength or expected strength) should be used in assessment. In the absence of reliable information, probable concrete strength can be obtained by $f'_{c,p} = 1.5 \times f'_c$, where fc' is the nominal strength of concrete. NZSEE 2006 Guidelines also include a brief explanation of the old concrete properties and strength; however, no detailed instructions to approximate the additional strength of the old concrete are provided.

In ASCE 41-13, it is defined that, under the circumstance where only limited information is available, the default concrete strengths specified in Table 3- 24 or Table 3- 25 can be applied, with the factors to translate from lower-bound material properties to the expected strength material properties specified in Table 3- 26. It is worth noting that the values of knowledge factor are determined based on the quality and quantity of the material data collected, and knowledge factors are applied in the assessment of component capacities, as discussed in Section 3.3.2.

In EN 1998-3: 2005 and NTC 2008, concrete strength obtained from design, construction documents, contemporary standards or testing results is suggested to use in assessment, and there are no instructions found to approximate the probable concrete strength in the absence of information. In Table 3- 27, the requirements regarding material testing are shown.

Table 3- 23: Determination of concrete strengths and properties in the four codified assessment procedures

Concrete	NZSEE 2006	ASCE 41-13	EN 1998-3: 2005	NTC 2008
Strength	(1) Testing (2) In the absence of reliable information: $f'_{c,p} = 1.5 \times f'_c$ $f'_{c,p}$=Probable concrete compressive strength f_c'=Nominal concrete compressive strength NOTE: old concrete tends to exceed the specified value. (No suggestion offered to deal with old concrete)	Table 4-2 (taken from Table 10-2, 10-3) for Tier 1 and maybe for Tier 2. No default values for E for concrete are provided. $E_c = 57000\sqrt{f'_c}\left(\frac{lb}{in^2}\right)$ for normalweight concrete Information required in accordance with Table 6-1 (knowledge level) for Tier 3	(1) Testing (from UNI codes) (destructive, semi destructive and non-destructive tests, but attention to the reliability of results) (2) Information from the code in force of the time of construction NOTE: non-destructive tests should not be used in isolation but in conjunction with destructive tests	(1) Testing (from UNI codes) (destructive, semi-destructive and non-destructive tests, but attention to the reliability of results) (2) Information from the code in force of the time of construction NOTE: It is possible to replace some destructive tests (not more than 50%) with a greater number (at least three times) of non-destructive ones. The non-destructive tests cannot substitute the destructive ones.
Ultimate Compressive Strain	<u>Unconfined concrete:</u> ε_{cu}=0.004 <u>Confined concrete:</u> $\varepsilon_{cu} = 0.004(1 + 1.1\rho_s f_{yt})$ ρ_s=ratio of volume of transverse reinforcement to volume of concrete core f_{yt}=probable yield strength of the transverse reinforcement Mander Model: $\varepsilon_{cu} = 0.004 + \frac{1.4\rho_s f_{yh}\varepsilon_{su}}{f_{cc}}$ $\rho_s = \frac{1.5A_v}{b_c s}$ A_v=total area of transverse reinforcement in a layer s=spacing of layers of transverse reinforcement b_c=width of column core, measured from centre to centre of the peripheral trans f_{yh}=yield strength of the transverse reinforcement ε_{su}=steel strain at maximum stress f_{cc}=compression strength of the confined concrete (e.g. $f_{cc}=1.5f_c'$)	Usable strain limits: <u>Without confining</u> with transverse reinforcement (at the extreme concrete compression fibre) $\varepsilon_{max} \leq 0.002$(for components in nearly pure compression) $\varepsilon_{max} \leq 0.005$ (for other components) (unless larger strains are substantiated by experimental evidence) <u>Confined concrete:</u> ε_{max} shall be based on experimental evidence and consider limitations posed by transverse reinforcement fracture, longitudinal reinforcement buckling, and degradation of component resistance at large deformation levels. $\varepsilon_{max} \leq 0.02$ (for longitudinal reinforcement in compression) $\varepsilon_{max} \leq 0.05$ (for longitudinal reinforcement in tension) Monotonic coupon test results shall not be used to determine reinforcement strain limits. If experimental evidence is used to determine strain limits, the effects of low-cycle fatigue and transverse reinforcement spacing and size shall be included in testing procedures.		

Table 3- 24: Default compressive strengths (fc') of structural concrete (unit: kip/in.^{2}) applied in Tier 1 evaluation (Table 4-2 from ASCE 41-13)

Time Frame	Beams	Slabs and Columns	Walls
1900–1919	2	1.5	1
1920–1949	2	2	2
1950–1969	3	3	2.5
1970–Present	3	3	3

Table 3- 25: Default lower-bound compressive strength of structural concrete (unit: lb/in.^{2}, MPa) applied in Tier 2 or 3 evaluation (Table 10-2 from ASCE 41-13)

Time Frame	Footings	Beams	Slabs	Columns	Walls
1900–1919	1000 to 2500 (7 to 17)	2000 to 3000 (14 to 21)	1500 to 3000 (10 to 21)	1500 to 3000 (10 to 21)	1000 to 2500 (7 to 17)
1920–1949	1500 to 3000 (10 to 21)	2000 to 3000 (14 to 21)	2000 to 3000 (14 to 21)	2000 to 4000 (14 to 28)	2000 to 3000 (14 to 21)
1950–1969	2500 to 3000 (17 to 21)	3000 to 4000 (21 to 28)	3000 to 4000 (21 to 28)	3000 to 6000 (21 to 40)	2500 to 4000 (17 to 28)
1970–present	3000 to 4000 (21 to 28)	3000 to 5000 (21 to 35)	3000 to 5000 (21 to 35)	3000 to 10,000 (21 to 70)	3000 to 5000 (21 to 35)

Table 3- 26: Factors to translate lower-bound material properties to expected strength material properties (Table 10-1 from ASCE 41-13)

Material Property	Factor
Concrete compressive strength	1.50
Reinforcing steel tensile and yield strength	1.25
Connector steel yield strength	1.50

Table 3- 27: Recommended minimum requirements for different levels of inspection and testing (EN 1998-3:2005 Table 3.2)

	Inspection (of details)	Testing (of materials)
	For each type of primary element (beam, column, wall):	
Level of inspection and testing	Percentage of elements that are checked for details	Material samples per floor
Limited	20	1
Extended	50	2
Comprehensive	80	3

It is worth noting that confinement of concrete has significant impact on concrete properties and strengths, as is shown in Table 3- 23. In NZSEE 2006, it is recommended that for unconfined concrete, an ultimate strain of 0.004 should be applied; and for confined concrete, Mander Model or other equivalent models should be adopted to approximate confined concrete properties and strengths. In ASCE 41-13, ultimate concrete strain is determined based on the past experimental evidences that are provided considering limitations imposed by transverse reinforcement fracture, longitudinal reinforcement buckling, and degradation of component resistance at large deformation levels. In EN 1998-3: 2005 and NTC 2008, there are no detailed instructions concerning the determination of confined concrete properties and strengths.

3.3.3.2. Reinforcing Steel

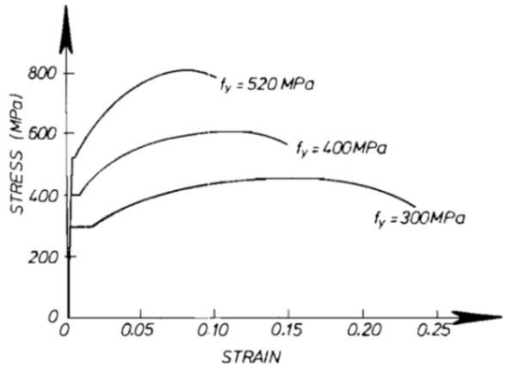


Figure 3- 19: Typical stress-strain curves for reinforcing steel (Paulay, T., Priestley, M.J.N., Seismic Design of Reinforced Concrete and Masonry Buildings)

Reinforcing steel properties and strengths considered in the assessment of reinforced concrete structures are listed as following, and are also illustrated in Figure 3- 19:

- Tensile yield stress (fsy) and yield strain (εsy)
- Tensile ultimate stress (fsu) and ultimate strain (εsu)
- Assumptions or models applied to approximate compressive properties based on tensile properties

Table 3- 28: Determination of steel strengths and properties in the four codified assessment procedures

Concrete	NZSEE 2006	ASCE 41-13	EN 1998-3: 2005	NTC 2008
Strength	(1) Testing (2) Apply the mean of the upper characteristic (95%) and lower characteristic (5%) , i.e. definition of mean strength (probable material strength, expected material strength) $f_{y,95\%} = (1.17\sim1.3) \times f_{y,5\%}$ Range of [1.17, 1.3] depends on source of information and age. (3) In the absence of reliable information: $f_{y,p} = 1.08 \times f_y$ $f_{y,p}$=Probable expected mean yield strength of reinforcing steels f_y=Lower characteristic yield strength of reinforcing steels, assumed to be equal to the nominal strength (4) For structures reinforced by structural grade reinforcement of the 1930-1970 period: (in the absence of information) $f_{y,p} = 300\text{MPa}$ (5) Steel history: SANZ (1962):$f_{y,min}$=227MPa (33000psi) SANZ (1963) & NZS1693: $f_{y,min}$=275MPa SANZ (1964): $f_{y,max}$=414MPa Chapman (1991): $f_{y,5\%,actual}$=(1+15%-20%)×$f_{y,5\%,specified}$ NOTE: plain round bars were used in NZ for longitudinal reinforcement until about the mid-1960s. Note for development length requirement (at least twice of that of deformed bars), more severe bond degradation thus greater stiffness reduction during cyclic loading.	Table 4-3, 4-4, 4-5 (taken from 10-3, 9-1, 9-2) for Tier 1 and maybe for Tier 2 $E_s = 29000(\text{kip/in}^2)$ $F_{pe} = 25\ (\text{kip})$ Information required in accordance with ASCE 41-13 Table 6-1 (knowledge level) for Tier 3	(1) Testing (from UNI codes) (destructive and not-destructive tests) (2) Information from the code in force of the time of construction NOTE: non-destructive tests should not be used in isolation but in conjunction with destructive tests	(1) Testing (from UNI codes) (destructive and not-destructive tests) (2) Information from the code in force of the time of construction NOTE: It is possible to replace some destructive tests (not more than 50%) with a greater number (at least three times) of non-destructive ones. The non-destructive tests cannot substitute the destructive ones
Strain	For older designs: ε_{su}=0.15 for f_y=275MPa ε_{su}=0.1 for f_y=430MPa			

As shown in Table 3- 28, in NZSEE 2006, if very limited information is available, it is suggested that probable reinforcing steel yield strength of 300MPa can be applied for the structures that were reinforced by structural grade steel from 1930 to 1970. It is also suggested that probable yield strength can be approximated by $f_{y,p} = 1.08 \times f_y$, where f_y is the lower characteristic yield strength of reinforcing steel, assumed to be equal to the nominal strength found in design and structural drawings. It is worth noting that there is lack of information of plain round bars which were used in New Zealand as longitudinal reinforcement until mid-1960s.In ASCE 41-13, as discussed in Section 3.2.2, different data requirements are specified corresponding to different knowledge levels. The following tables (Table 3- 29 to Table 3- 32) provide summarises of the reinforcing steel default yield strengths, lower-bound tensile and yield properties, and factors to translate lower-bound steel properties to expected-strength steel properties. In EN1998-3: 2005 and NTC 2008, it is recommended that reinforcing steel properties should be obtained from physical testing or design standards.

Table 3- 29: Default yield strength (f_y) of reinforcing steel (unit: kip/in^2) (Table 4-3 from ASCE 41-13)

		Structural[a]	Intermediate[a]	Hard[a]				
	Grade	33	40	50	60	65	70	75
Year	Minimum Yield[a] (kip/in.^{2})	33	40	50	60	65	70	75
1911–1959		X	X	X		X		
1959–1966		X	X	X	X	X	X	X
1966–1987			X	X	X	X	X	
1987–present			X	X	X	X	X	X

Table 3- 30: Default lower-bound tensile and yield properties of reinforcing steel(Table 10-3 ASCE 41-13)

		Structural[a]	Intermediate[a]	Hard[a]				
	Grade	33	40	50	60	65	70	75
Year	Minimum Yield, lb/in.^{2} (MPa) Minimum Tensile, lb/in.^{2} (MPa)	33,000 (230) 55,000 (380)	40,000 (280) 70,000 (485)	50,000 (350) 80,000 (550)	60,000 (420) 90,000 (620)	65,000 (450) 75,000 (520)	70,000 (485) 80,000 (550)	75,000 (520) 100,000 (690)
1911–1959		x	x	x		x		
1959–1966		x	x	x	x	x	x	x
1966–1972			x	x	x	x	x	
1972–1974			x	x	x	x	x	
1974–1987			x	x	x	x	x	
1987–present			x	x	x	x	x	x

Table 3- 31: Default lower-bound tensile and yield properties of reinforcing steel for various ASTM specifications (Table 10-4 from ASCE 41-13)

				Structural[a]	Intermediate[a]	Hard[a]				
			ASTM Grade	33	40	50	60	65	70	75
			Minimum Yield, lb/in.^{2} (MPa) Minimum Tensile, lb/in.^{2} (MPa)	33,000 (230) 55,000 (380)	40,000 (280) 70,000 (485)	50,000 (350) 80,000 (550)	60,000 (420) 90,000 (620)	65,000 (450) 75,000 (520)	70,000 (485) 80,000 (550)	75,000 (520) 100,000 (690)
ASTM Designation[b]	Steel Type	Year Range								
A15 (withdrawn)	Billet	1911–1966		x	x	x				
A16 (withdrawn)	Rail[c]	1913–1966				x				
A61 (withdrawn)	Rail[c]	1963–1966					x			
A160 (withdrawn)	Axle	1936–1964		x	x	x				
A160 (withdrawn)	Axle	1965–1966		x	x	x	x			
A185	WWR	1936–present						x		
A408 (withdrawn)	Billet	1957–1966		x	x	x				
A431	Billet	1959–1966								x
A432 (withdrawn)	Billet	1959–1966					x			
A497	WWR	1964–present							x	
A615/A615M (2003c)	Billet	1968–1972			x		x			x
A615/A615M (2003c)	Billet	1974–1986			x		x			
A615/A615M (2003c)	Billet	1987–present			x		x			x
A616[d] (withdrawn)	Rail[c]	1968–present				x	x			
A617 (withdrawn)	Axle	1968–present			x		x			
A706/ A706M[e]	Low-alloy	1974–present					x			
A955	Stainless	1996–present			x		x			x

Table 3- 32: Factors to translate lower-bound steel properties to expected-strength steel properties (Table 9-3 from ASCE 41-13)

Property	Year	Specification	Factor
Tensile strength	Before 1961		1.10
Yield strength	Before 1961		1.10
Tensile strength	1961–1990	ASTM A36	1.10
	1961–Present	ASTM A572, Group 1	1.10
		ASTM A572, Group 2	1.10
		ASTM A572, Group 3	1.05
		ASTM A572, Group 4	1.05
		ASTM A572, Group 5	1.05
	1990–Present	ASTM A36 and Dual Grade, Group 1	1.05
		ASTM A36 and Dual Grade, Group 2	1.05
		ASTM A36 and Dual Grade, Group 3	1.05
		ASTM A36 and Dual Grade, Group 4	1.05
	1998–Present	ASTM A992	1.10
Yield strength	1961–1990	ASTM A36	1.10
	1961–Present	ASTM A572, Group 1	1.10
		ASTM A572, Group 2	1.10
		ASTM A572, Group 3	1.05
		ASTM A572, Group 4	1.10
		ASTM A572, Group 5	1.05
	1990–Present	ASTM A36, Plates	1.10
		ASTM A36 and Dual Grade, Group 1	1.05
		ASTM A36 and Dual Grade, Group 2	1.10
		ASTM A36 and Dual Grade, Group 3	1.05
		ASTM A36 and Dual Grade, Group 4	1.05
	1998–Present	ASTM A992	1.10
Tensile strength	All	Not listed[a]	1.10
Yield strength	All	Not listed[a]	1.10

[a]For materials not conforming to one of the listed specifications.

3.3.4. Assessment at Component Level

Table 3- 33: General requirements regarding the determination of component properties and strengths

Component	NZSEE 2006	ASCE 41-13	EN 1998-3: 2005	NTC 2008
General	Probable strength: Theoretical strength should be calculated based on established theory. Section dimensions and mean material strengths are required in the calculation.	(1) Cross-sectional dimensions of individual components and overall configuration of the structure; (2) Configuration of component connections, size of anchor bolts, thickness of connector material, anchorage and interconnection of embedment and the presence of bracing or stiffening components; (3) Modifications to components or overall configuration of the structure; (4) Most recent physical condition of components and connections, and the extent of any deterioration; (5) Deformations beyond those expected because of gravity loads, such as those caused by settlement or past earthquake events; (6) Presence of other conditions that influence building performance, such as nonstructural components that may interfere with structural components during earthquake excitation. The analytical model for a beam–column frame element shall represent strength, stiffness, deformation capacity of beams, columns, joints, and other components (e.g. slabs), and including connections with other elements .Potential failure in flexure, shear, and reinforcement development at any section along the component length shall be considered. Interaction with other elements, including non-structural components, shall be included. The analytical model for a shear wall element shall represent the stiffness, strength, and deformation capacity of the shear wall. Potential failure in flexure, shear, and reinforcement development at any point in the shear wall shall be considered. Interaction with other structural and nonstructural components shall be included. **Stiffness:** considering shear, flexure, axial behaviour, reinforcement slip deformation, stress state, cracking extent caused by volumetric changes from temperature and shrinkage, and deformation levels under gravity loads and seismic forces **Strength:** maximum component strength shall be determined considering potential failure in flexure, axial load, shear, torsion, bar development, and other actions at all points along the length of the component, under the actions of design gravity load and seismic force combinations.	Overall dimensions and cross-sectional properties of the buildings elements; Amount of longitudinal steel in beams, columns and walls; amount and detailing of confining steel in critical regions and in beam- column joints; amount of steel reinforcement in floor slabs contributing to the negative resisting bending moment of T-beams; seating lengths and support conditions of horizontal elements; depth of concrete cover; lap-splices for longitudinal reinforcement NOTE: this information is provided by simulated design in accordance with relevant practice, original detailed construction drawings and in-situ inspections	Geometrical dimensions of structural elements and quantity of reinforcement; Possible defects in the detailing (reinforcements details, beam-column eccentricity, column-column eccentricity, beam-column and column-foundation connection, etc.), amount of reinforcement (spacing and other details), connection between steel members, contribution of non-structural components NOTE: this information is provided by simulated design in accordance with relevant practice, original detailed construction drawings and in-situ inspections

Table 3- 33 provides a summary of general requirements of input data to determine component strengths and properties specified in the four codified assessment procedures. The details associated with assessment of beams, columns, beam-column joints, and shear walls, are shown from Section 3.3.4.1 to Section 3.3.4.4.

NZSEE 2006 suggests that probable flexural strength of components (beams, columns, walls, etc.) should be used in assessment. Either hand calculation or computer section analysis (e.g. RESPONSE2000, ABSTRACT, etc.) can be carried out to determine component flexure and shear strength, and it is worth noting that the calculation or analysis procedures are referred to SNZ1995 (i.e. NZS3101:1995). It is worth noting that NZSEE 2006 defines limit state criteria at global structure level rather than at component level, i.e. it defines a 2.5% interstorey drift corresponding to life safety limit state, while the other three codified procedures provide detailed specifications of limit state criteria at component level.

In EN 1998-3:2005 and NTC 2008, procedures and formulae to compute the flexural and shear strength for components are specified at different limit states (Damage Limitation, Significant Damage, and Near Collapse limit states for EN 1998-3:2005; Serviceability, Life Safeguard, and Collapse limit states for NTC 2008) and for different predicted mechanisms (i.e. ductile or brittle).

ASCE 41-13, however, different from the other three codified assessment procedures, adopts force-deformation models (with numerical acceptance criteria) for components such as beams, columns, beam-column joints, walls, slabs, infills, and so on. In Figure 3- 20 and Figure 3- 21, component force-deformation relationships (i.e. component analysis models) with acceptance criteria at different performance levels are shown. Summary tables (Table 3- 35, Table 3- 36, Table 3- 38, Table 3- 41, Table 3- 43, Table 3- 45, Table 3- 47, Table 3- 48, Table 3- 50, Table 3- 51, and for other components, see ASCE 41-13), in which the values specified for modelling parameters (applied in linear or nonlinear analyses) and numerical acceptance criteria (i.e. component plastic rotation angle, etc.) are recorded, are provided by ASCE 41-13. As shown in Figure 3- 21 and the summary tables, ASCE 41-13 specifies plastic rotation angles or drifts corresponding to Immediate Occupancy, Life Safety and Collapsed Prevention performance levels, and these numerical criteria are usually directly adopted in computer analysis programs or tools. More discussions associated with the establishment and application of such component models are in Section 5.4.3.

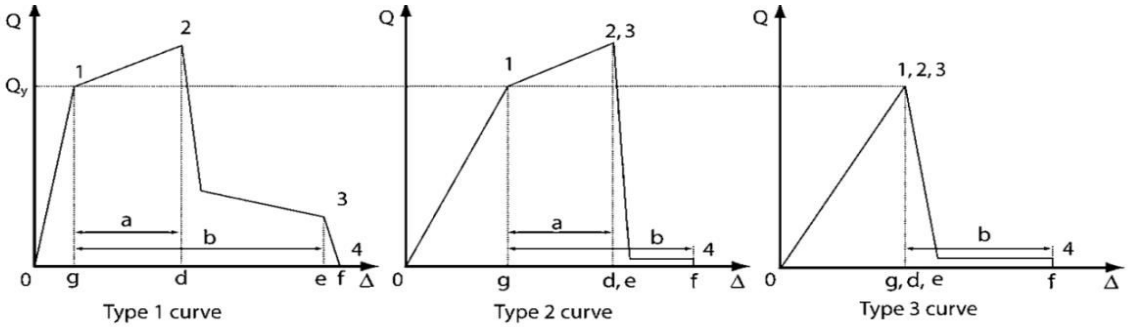


Figure 3- 20: Component force versus deformation curves (ASCE41-13 Figure 7-4)
(Note: Only secondary component actions permitted between points 2 and 4)

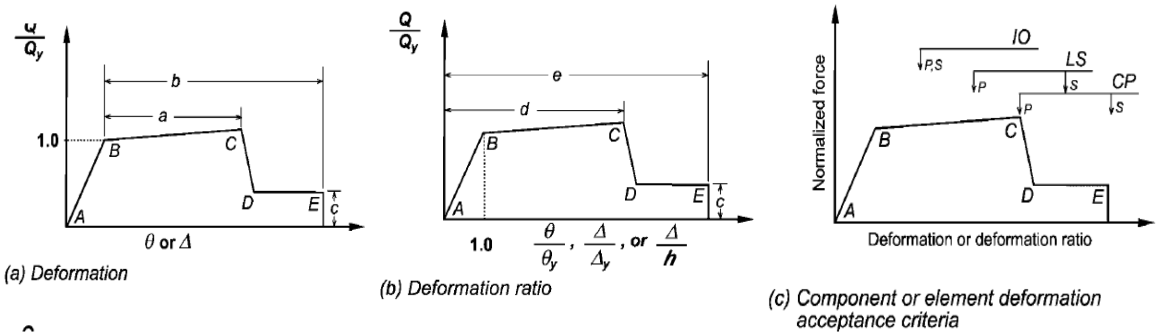


Figure 3- 21: Generalised component force-deformation or normalised force-deformation ratio relations for depicting modelling and acceptance criteria (ASCE 41-13, Commentary)

It can be concluded that ASCE 41-13 provides much more comprehensive instructions regarding the determination of material and component properties or strengths, compared to the other three procedures. The followings are clearly specified in ASCE 41-13 while are not specifically included in the other codified procedures.

- Very detailed instructions associated with material strengths to be applied, for instances, tables of default values and lower characteristic values along the American design or testing history, etc.
- Detailed instructions associated with the material testing required
- Force-deformation models for a variety of components, i.e. the component models
- Detailed instruction associated with the assessment of nonstructural elements

3.3.4.1. Beams

Table 3- 34 provide a summary of procedures to determine beam strengths from the four codified assessment procedures.

As specified in NZSEE 2006, beam flexural strength should be estimated (as previously mentioned, either by hand calculation or computer section analysis programs) considering contribution of slab reinforcements, bond deterioration, bond slip effect, etc. Beam rotation capacities can then be assessed based on the computation of yield curvature, ultimate curvature and plastic hinge length. Additionally, the upper bound and the lower bound of the mean flexural strength can be determined as beam overstrength and nominal strength, respectively. The determination of beam shear strength is referred to design code SNZ1995 (i.e. NZS3101:1995), taking strength degradation due to cyclic loading into consideration.

As specified in EN1998-3:2005 and NTC 2008, different criteria should be applied depending on the predicted mechanisms of the beam. For instance, if a ductile mechanism is expected, beam rotation capacity corresponding to different limit states should be assessed, and then need to be compared to the acceptance criteria in terms of rotation. However, if a brittle mechanism is expected, beam shear strength under different limit states should be estimated, and then should be compared to the acceptance criteria in terms of shear force.

In ASCE 41-13, beam force-deformation models are adopted. In Table 3- 35 and Table 3- 36, information regarding modelling parameters and numerical acceptance criteria applied in nonlinear and linear analyses of beams is gathered. The details associated with analysis approaches are presented in Chapter 4.

Table 3- 34: Determination of beam strengths/capacities

Beam	NZSEE 2006	ASCE 41-13	EN 1998-3: 2005	NTC 2008
Strengths/ Capacities	**Flexural strength:** Nominal flexural strength (lower bound) Probable flexural strength: $M_{b,p} = 1.08 \times M_{b,n}$ $M_{b,p}$=Beam probable flexural strength $M_{b,n}$=Beam nominal flexural strength Flexural overstrength (upper bound) (1) Overstrength of the member is mainly due to the steel properties. Use the upper bound (95%) steel yield strength to determine beam flexural overstrength (2) $M_{b,o} = 1.25 \times M_{b,n}{}^{(*)}$ Due to a further 8% increase in steel stress due to strain hardening (*) Currently assumed in NZ for both Grade 300 and Grade 430 steel <u>Cast-in-place floor</u> slabs integrally built with the beams: reinforcement in slabs should be included, and check slab width and bar anchorage to ensure adequate tensile strength can be developed. <u>T- and L- beams</u> (built integrally with slabs): check for slab width, anchorage,	<u>Stiffness:</u> Considering flexural and shear stiffness, incl. effect of the slab acting as a flange in monolithic construction, additional flexibility from reinforcement slip within beam-column joint Table 10-5 <u>Strength:</u> Table 10-7 and 10-13	**Ductile mechanism – under flexure:** <u>NC limit state:</u> total chord rotation capacity at ultimate (elastic + inelastic part) : $\theta_{um} = \frac{1}{\gamma_{el}}\left(\theta_y + (\varphi_u - \varphi_y)L_{pl}\left(1 - \frac{0.5L_{pl}}{L_v}\right)\right)$ $\gamma_{el} = 1.5$ for primary elements and 1 for secondary elements L_v = shear span Depending on how the enhancement of strength and deformation capacity of concrete due to confinement is taken into account in the calculation of ultimate curvature: $L_{pl} = 0.1L_v + 0.17h + \frac{0.24d_{bL}f_y}{\sqrt{f_c}}$	**Ductile mechanism – under flexure:** <u>Collapse limit state</u> total chord rotation capacity at ultimate (elastic + inelastic part) :$\theta_{um} = \frac{1}{\gamma_{el}}\left(\theta_y + (\varphi_u - \varphi_y)L_{pl}\left(1 - \frac{0.5L_{pl}}{L_v}\right)\right)$ $\gamma_{el} = 1.5$ for primary elements and 1 for secondary elements L_v : shear span ϕ_u : Ultimate curvature considering the ultimate strain of concrete (considering the confinement) and steel (without information the

	contribution of slab reinforcement Bond deterioration due to cyclic loading, up 10% decrease for positive moment and up to 5% for negative moment for beam flexural strength (Hakuto*et al 1999)* Bond slip effect: negligible to flexural strength of beams (unlikely that that will be a total loss of bond unless plain round bars are present) Bounds of flexural strength of beam: range of expected material strengths should be considered when estimating maximum and minimum likely expected flexural strengths **Shear strength:** (The strength reduction factor of 0.85 has been built into the formulae): $V_{BPI} = 0.85\left(v_c b_w d + \frac{A_v f_{yt} d}{s}\right) = 0.85\left(k\sqrt{f_c'} b_w d + \frac{A_v f_{yt} d}{s}\right)$ SNZ (1995): (conservative estimate) $k=0.07+10p_w=0.07+10As/(b_w d)$ As=area of tension reinforcement $0.08 \leq k \leq 0.2$ (It was suggested that k=0.2 could be assumed for beams without plastic hinging, based on test results by Hakuto *et al* (1995) and Priestley (1995). Note that k=0.2 is conservative for high longitudinal steel contents.) Degradation of shear strength: (degradation of the nominal shear stress carried by the concrete, $k\sqrt{f_c'}$) **Ductility capacity: (curvature/ rotation)** First yield curvature (1) $\varphi_y = \frac{\varepsilon_y}{d-kd}$ ε_y = strain at first yield of the longitudinal tension reinforcement d = effective depth of longitudinal tension reinforcement kd = neutral axial depth when tension steel reaches the strain at first yield (2) $\varphi_y = \frac{1.7\varepsilon_y}{h}$ where h=beam depth (Priestley and Kowalsky, 2000) Ultimate curvature: $\varphi_u = \frac{\varepsilon_{cu}}{c}$ Equivalent plastic hinge length (1) Lp=0.5h (2) $Lp=0.08L+0.022fyd_b$ L = distance of the critical plastic hinge section from the estimated point of contra-flexure (L=0.5Lc, where Lc=beam clear span) fy = probable yield strength of longitudinal reinforcement d_b = diameter of longitudinal reinforcement Rotation capacity (=> plastic story drift in a beam sidesway mechanism): $\theta_p = (\varphi_u - \varphi_y)\, L_p$		$L_{pl} = \frac{L_v}{30} + 0.2h + \frac{0.11 d_{bL} f_y}{\sqrt{f_c}}$ d_{bL}= mean diameter of the tension reinforcement Other empirical expressions can be used for the calculation of *θum* and θ_{um}^{pl}. SD limit state: *θSD = 3/4θum* DL limit state: chord rotation capacity at yielding: $\theta_y = \phi_y \frac{L_v + a_V z}{3} + 0.00135\left(1 + \frac{1.5h}{L_v}\right) + \frac{\varepsilon_y d_b f_y}{(d-d')\cdot 6\sqrt{f_c}}$ $a_V z$ = the tension shift of the bending moment diagram ($a_V = 1$ if $M_y > L_v V_{R,c}$ otherwise $a_V = 0$ if $M_y < L_v V_{R,c}$) Or from the alternative expressions: $\theta_y = \phi_y \frac{L_v + a_V z}{3} + 0.0013\left(1 + \frac{1.5h}{L_v}\right) + \frac{0.13\phi_y d_b f_y}{\sqrt{f_c}}$ **Brittle mechanism – shear:** NC limit state: $V_R = \frac{1}{\gamma_{el}}\left[\frac{h-x}{2L_v}\min(N; 0.55A_c f_c) + \left(-0.05\min\left(5; \mu_\Delta^{pl}\right)\right)\cdot \cdot\left[0.16\max(0.5; 100\rho_{tot})\cdot \cdot\left(1 - 0.16\min\left(5; \frac{L_v}{h}\right)\right)\sqrt{f_c}A_c + V_w\right]\right]$ *VR* decreases with the plastic part of the ductility demand x = compression zone depth Vw = contribution of transverse reinforcement For cross-sections with rectangular web of width bw: $V_w = \rho_w b_w z f_{yw}$ For circular cross-sections: $V_w = \frac{\pi}{2}\frac{A_{sw}}{s} f_{yw}(D - 2c)$ The minimum of shear resistance calculated in accordance with EN1992-1-1:2004 or by means of the previous expressions should be used in the assessment SD and DL limit states: The verification is not required unless these LS are the only ones to be checked (in that case see NC)	maximum strain can be assumed equal to 4%) $L_{pl} = 0.1L_v + 0.17h + \frac{0.24 d_{bL} f_y}{\sqrt{f_c}}$ d_{bL}= mean diameter of the tension reinforcement Another empirical expression can be used for the calculation of *θu.* Life safeguard limit state: *θSD = 2/3θu* Serviceability limit state: chord rotation capacity at yielding $\theta_y = \frac{\phi_y L_v}{3} + 0.0013\left(1 + \frac{1.5h}{L_v}\right) + \frac{0.13\phi_y d_b f_y}{\sqrt{f_c}}$ f_c and f_y are obtained as average from in situ tests divided by partial safety factor and CF **Brittle mechanism – shear:** Ultimate limit states: $V_{Rd} = \left[0.18 \cdot k \cdot \frac{(100\rho_1 f_{ck})^{\frac{1}{3}}}{\gamma_c} + 0.15 \cdot \sigma_{cp}\right] \cdot b_w d \geq (v_{min} + 0.15 \cdot \sigma_{cp}) b_w d$ $k = 1 + \left(\frac{200}{d}\right)^{\frac{1}{2}} \leq 2$ $v_{min} = 0.035 k^{\frac{3}{2}} f_{ck}^{\frac{1}{2}}$ d=useful height of section $\rho_l = \frac{A_{sl}}{(b_w d)} \leq 0.02$ $\sigma_{cp} = \frac{N_{Ed}}{A_c} \leq 0.2 f_{cd}$ b_w=minimum width of section Material strength are average from in situ tests and other information divided by CF and partial safety factor

Table 3- 35: Modelling parameters and numerical acceptance criteria for reinforced concrete beams for nonlinear procedures (ASCE 41-13, Table 10-7)

Conditions			Modeling Parameters[a]			Acceptance Criteria[a]		
			Plastic Rotations Angle (radians)		Residual Strength Ratio	Plastic Rotations Angle (radians)		
						Performance Level		
			a	b	c	IO	LS	CP
Condition i. Beams controlled by flexure[b]								
$\frac{\rho-\rho'}{\rho_{bal}}$	Transverse reinforcement[c]	$\frac{V}{b_w d\sqrt{f'_c}}$ [d]						
≤0.0	C	≤3 (0.25)	0.025	0.05	0.2	0.010	0.025	0.05
≤0.0	C	≥6 (0.5)	0.02	0.04	0.2	0.005	0.02	0.04
≥0.5	C	≤3 (0.25)	0.02	0.03	0.2	0.005	0.02	0.03
≥0.5	C	≥6 (0.5)	0.015	0.02	0.2	0.005	0.015	0.02
≤0.0	NC	≤3 (0.25)	0.02	0.03	0.2	0.005	0.02	0.03
≤0.0	NC	≥6 (0.5)	0.01	0.015	0.2	0.0015	0.01	0.015
≥0.5	NC	≤3 (0.25)	0.01	0.015	0.2	0.005	0.01	0.015
≥0.5	NC	≥6 (0.5)	0.005	0.01	0.2	0.0015	0.005	0.01
Condition ii. Beams controlled by shear[b]								
Stirrup spacing ≤ $d/2$			0.0030	0.02	0.2	0.0015	0.01	0.02
Stirrup spacing > $d/2$			0.0030	0.01	0.2	0.0015	0.005	0.01
Condition iii. Beams controlled by inadequate development or splicing along the span[b]								
Stirrup spacing ≤ $d/2$			0.0030	0.02	0.0	0.0015	0.01	0.02
Stirrup spacing > $d/2$			0.0030	0.01	0.0	0.0015	0.005	0.01
Condition iv. Beams controlled by inadequate embedment into beam–column joint[b]								
			0.015	0.03	0.2	0.01	0.02	0.03

NOTE: f'_c in lb/in.[2] (MPa) units.
[a]Values between those listed in the table should be determined by linear interpolation.
[b]Where more than one of conditions i, ii, iii, and iv occur for a given component, use the minimum appropriate numerical value from the table.
[c]"C" and "NC" are abbreviations for conforming and nonconforming transverse reinforcement, respectively. Transverse reinforcement is conforming if, within the flexural plastic hinge region, hoops are spaced at ≤ $d/3$, and if, for components of moderate and high ductility demand, the strength provided by the hoops (V_s) is at least 3/4 of the design shear. Otherwise, the transverse reinforcement is considered nonconforming.
[d]V is the design shear force from NSP or NDP.

Table 3- 36: Numerical acceptance criteria for reinforced concrete beams for linear proceudres (ASCE 41-13, Table 10-13)

Conditions			m-Factors[a]				
			Performance Level				
				Component Type			
				Primary		Secondary	
			IO	LS	CP	LS	CP
Condition i. Beams controlled by flexure[b]							
$\frac{\rho-\rho'}{\rho_{bal}}$	Transverse reinforcement[c]	$\frac{V}{b_w d\sqrt{f'_c}}$ [d]					
≤0.0	C	≤3 (0.25)	3	6	7	6	10
≤0.0	C	≥6 (0.5)	2	3	4	3	5
≥0.5	C	≤3 (0.25)	2	3	4	3	5
≥0.5	C	≥6 (0.5)	2	2	3	2	4
≤0.0	NC	≤3 (0.25)	2	3	4	3	5
≤0.0	NC	≥6 (0.5)	1.25	2	3	2	4
≥0.5	NC	≤3 (0.25)	2	3	3	3	4
≥0.5	NC	≥6 (0.5)	1.25	2	2	2	3
Condition ii. Beams controlled by shear[b]							
Stirrup spacing ≤ $d/2$			1.25	1.5	1.75	3	4
Stirrup spacing > $d/2$			1.25	1.5	1.75	2	3
Condition iii. Beams controlled by inadequate development or splicing along the span[b]							
Stirrup spacing ≤ $d/2$			1.25	1.5	1.75	3	4
Stirrup spacing > $d/2$			1.25	1.5	1.75	2	3
Condition iv. Beams controlled by inadequate embedment into beam–column joint[b]							
			2	2	3	3	4

NOTE: f'_c in lb/in.[2] (MPa) units.
[a]Values between those listed in the table should be determined by linear interpolation.
[b]Where more than one of conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.
[c]"C" and "NC" are abbreviations for conforming and nonconforming transverse reinforcement. Transverse reinforcement is conforming if, within the flexural plastic hinge region, hoops are spaced at ≤ $d/3$, and if, for components of moderate and high ductility demand, the strength provided by the hoops (V_s) is at least 3/4 of the design shear. Otherwise, the transverse reinforcement is considered nonconforming.
[d]V is the design shear force calculated using limit-state analysis procedures in accordance with Section 10.4.2.4.1.

3.3.4.2. Columns

Table 3- 37 provides a summary of procedures to determine column strengths from the four codified assessment procedures.

In NZSEE 2006, similar to the determination of beam strength, it is specified that column flexural strength should be estimated considering bond deterioration, slip effect and effect of earthquake induced axial load, with the upper and lower bounds approximated. The determination of column shear strength is referred to design code SNZ 1995 (i.e. NZS3101:1995), taking degradation of shear strength due to cyclic loading into consideration.

In EN1998-3:2005 and NTC 2008, the determination of column capacities – flexural rotation capacity or shear strength, is similar to the determination of beam capacities. EN 1998-3:2005 specifies additional guidance regarding column crushing failure.

In ASCE 41-13, similar to the beam force-deformation models, column force-deformation models are adopted. In Table 3- 38 and Table 3- 41, information regarding modelling parameters and numerical acceptance criteria applied in nonlinear and linear analyses of columns is gathered. The details associated with analysis approaches are presented in Chapter 4.

Table 3- 37: Determination of column strengths/capacities

Column	NZSEE 2006	ASCE 41-13	EN 1998-3:2005	NTC 2008
Strengths/ Capacities	**Flexural strength:** Flexural overstrength (upper bound) For columns confined by NZS3101:1995 specified amount of transverse reinforcement in potential plastic hinge regions: $M_{col,o} = \left(1.25 + 2\left(\frac{N^*}{f'_c A_g} - 0.1\right)^2\right) M_{col,n} \geq 1.25 \times M_{col,n}$ For columns with less confining reinforcement than currently specified in potential plastic hinge regions: the expected overstrength material strengths should be used to calculated the column flexural overstrength Bond slip effect: check for lap length ($\geq 20d_b$ for deformed bars), bond strength capacity degradation. $M_f = \max[\frac{l_{lap}}{l_d} M_n, 0.5N(D-a)]$ $M_{lap} = M_n - \frac{\theta_p}{0.025}(M_n - M_f)$ M_f=Final moment capacity M_n=Initial full moment capacity M_{lap}=Moment capacity of a lap splice l_{lap}=provided lap length l_d=theoretical development length D=overall width of the member a=depth of the compression stress block θ_p=plastic rotation demand on the connection (Residual strength remains after the lap splice fails in bond, due to the presence of eccentric compression stress block, preventing a catastrophic failure) Earthquake induced axial forces: for	**Stiffness:** Considering flexural, shear and axial stiffness, additional flexibility from reinforcement slip within beam-column joint or foundation before yielding Table 10-5 NOTE: low axial load **Strength:** Table 10-8 and 10-9 (SAME for beams and Walls) **10.3.5 Development and Splices of Reinforcement** Development of straight bars, hooked bars, and lap-spliced bars shall be calculated according to the provisions of ACI 318, with the following modifications: 1. Deformed straight, hooked, and lap-spliced bars shall meet the development requirements of Chapter 12 of ACI 318, except for lap splices, which shall be the same as those for straight development of bars in tension without consideration of lap-splice classifications; 2. Where existing deformed straight bars, hooked bars, and lap-spliced bars do not meet the development requirements of (1) above, the capacity of existing reinforcement shall be calculated using Eq. (10-1): $f_s = 1.25\left(\frac{l_b}{l_d}\right)^{2/3} f_{yL}$ (10-1) but shall not exceed the expected or lower-bound yield strength, as applicable. Where transverse reinforcement is distributed along the development length with spacing not exceeding one-third of the effective component depth, it shall be permitted to assume that the reinforcement retains the calculated maximum stress to high ductility demands. For larger spacings of transverse reinforcement, the developed stress shall be assumed to degrade from $1.0f_s$, at a ductility demand or DCR equal to 1.0, to $0.2f_s$ at a ductility demand or DCR equal to 2.0; 3. Strength of deformed straight, discontinuous bars embedded in concrete sections or beam–column joints, with clear cover over the embedded bar not less than $3d_b$, shall be calculated according to Eq. (10-2): $f_s = \frac{2500}{d_b} l_e \leq f_y$ (lb/in.² units) $f_s = \frac{17}{d_b} l_e \leq f_y$ (MPa units) (10-2)	**Ductile mechanism – under flexure:** same as beam capacity **Brittle mechanism – shear:** same as beam capacity Add: If in a concrete column the shear span ratio, $\frac{L_V}{h}$, at the end section with the maximum of the two end moments, is less or equal to 2.0, its shear strength V_R should not be taken greater than the value corresponding to failure by web crushing along the diagonal of the column after flexural yielding, $V_{R,max}$: $V_{R,max} = \frac{\frac{4}{7}(1 - 0.02\min(5;\mu_\Delta^{pi})}{\gamma_{el}}$ $+ 1.35\frac{N}{A_c f_c}\Big)\cdot$ $\cdot(1 + 0.45(100\rho_{tot}))\sqrt{\min(40;f_c)}\, b_w z$ γ_{el}=1.15 for primary seismic elements and 1.0 for secondary seismic ones	**Ductile mechanism – under flexure:** same as beam capacity **Brittle mechanism – shear:** same as beam capacity

multi-bay frames, the earthquake induced axial forces are not significant in comparison to gravity actions. In order to avoid running a frame analysis at this early stage (i.e. before the available displacement ductility is ascertained),for critical corner columns the beam shear capacities from the end spans can be summed and factored by Rv from Appendix A of SNZ (1995) as an initial approximation.

<u>Bounds of flexural strength of column:</u>
range of expected material strengths should be considered when estimating maximum and minimum likely expected flexural strengths

Shear strength: (SNZ1995, conservative estimate)
(The strength reduction factor of 0.85 has been built into the formulae, another factor of 0.85 applied to obtain a closer estimate of the lower bound test data)

$$V_{CPI} = 0.72(V_C + V_s + V_n)$$
$$V_C = v_c 0.8A_g = k\sqrt{f'_c}0.8A_g$$
$$V_s = \frac{A_v f_{yt} d''}{s}\cot 30^\circ \text{ rectangular hoop}$$
$$V_s = \frac{\pi}{2}\frac{A_{sp} f_{yt} d''}{s}\cot 30^\circ \text{ circular hoop}$$
$$V_n = N^* \tan\infty$$

<u>Degradation of shear strength:</u>
(degradation of the nominal shear stress carried by the concrete, $k\sqrt{f'_c}$ (the difference between the magnitudes of the shear resisted by the concrete mechanisms for beams and columns is attributed to the distributed longitudinal reinforcement of columns.)

Ductility capacity: (curvature/rotation)
First yield curvature
(1) Use a bilinear approximation of column moment curvature response, since there is no well-defined yield curvature for column.
(2) $\varphi_y = \frac{2.35\varepsilon_y}{D}$ where D=column diameter
$\varphi_y = \frac{2.12\varepsilon_y}{h}$ where h=column depth (Priestley and Kowalsky, 2000)

Ultimate curvature: $\varphi_u = \frac{\varepsilon_{cu}}{c}$

Equivalent plastic hinge length
(1) Lp=0.5h
(2) Lp=0.08L+0.022fyd_b
L = distance of the critical plastic hinge section from the estimated point of contra-flexure
fy = probable yield strength of longitudinal reinforcement
d_b = diameter of longitudinal reinforcement

Since axial load critically affects the ultimate curvature, it is essential that seismic axial forces be included when estimating column plastic rotation. The critical column will be the one with highest axial compression.

where f_s is less than f_y, and the calculated stress in the bar caused by design loads equals or exceeds f_s, the maximum developed stress shall be assumed to degrade from $1.0f_s$ to $0.2f_s$ at a ductility demand or DCR equal to 2.0. In beams with short bottom bar embedments into beam–column joints, flexural strength shall be calculated considering the stress limitation of Eq. (10-2);
4. For plain straight, hooked, and lap-spliced bars, development and splice lengths shall be taken as twice the values determined in accordance with ACI 318, unless other lengths are justified by approved tests or calculations considering only the chemical bond between the bar and concrete; and
5. Doweled bars added in seismic retrofit shall be assumed to develop yield stress where all the following conditions are satisfied:
 a. Drilled holes for dowel bars are cleaned with a stiff brush that extends the length of the hole;
 b. Embedment length l_e is not less than $10d_b$ and;
 c. Minimum dowel bar spacing is not less than $4l_e$ and minimum edge distance is not less than $2l_e$.

Shear strength:

$$V_n = kV_o = k\left[\frac{A_v f_y d}{s} + \lambda\left(\frac{6\sqrt{f'_c}}{M/Vd}\sqrt{1+\frac{N_u}{6\sqrt{f'_c}A_g}}\right)0.8A_g\right]$$
(lb/in²)

$$V_n = kV_o = k\left[\frac{A_v f_y d}{s} + \lambda\left(\frac{0.5\sqrt{f'_c}}{M/Vd}\sqrt{1+\frac{N_u}{0.5\sqrt{f'_c}A_g}}\right)0.8A_g\right] \text{(MPa)}$$

k = 1.0 in regions where displacement ductility demand is less than or equal to 2
k = 0.7 in regions where displacement ductility is greater than or equal to 6, and varies linearly for displacement ductility between 2 and 6
λ = 0.75 lightweight aggregate concrete
λ = 1.0 normal weight aggregate concrete
N_u = axial compression force (0 for tension)
M/Vd = largest ratio of moment to shear times effective depth under design loadings for the column, 2 < M/Vd< 4
d = effective depth (d=0.8h, h=dimension of the column in the direction of shear)
Ag = gross cross-sectional area of column
NOTE:
(1) Calculation of N_u: (from linear procedures) Max N_u considering design gravity load only, and Min N_u considering both gravity and earthquake loading
(2) Alternative formulations for column strength that consider effects of reversed cyclic inelastic deformations and that are verified by experimental evidence shall be permitted
(3) For columns satisfying detailing and proportioning requirements of ACI 318 Chapter 21, the shear strength equations of ACI 318 shall be permitted.
(4) k values and displacement ductility demand (reduction in column shear capacity with increasing nonlinear deformations)
For a column experiencing flexural yielding before shear failure (Vp< Vo), displacement ductility demand is defined as $\frac{\text{maximum displacement demand}}{\text{yield displacement}}$
The yield displacement is the lateral displacement of the column, determined using the effective rigidities from Table 10-5, at a shear demand resulting in flexural yielding of the plastic hinges, Vp. The maximum displacement demand for the column can be estimated as the maximum interstorey displacement demand. Alternatively, the interstorey displacement demand can be refined by accounting for the interstorey displacements caused by rigid body rotations at the column's base and top.

Table 3- 38: Modelling parameters and numerical acceptance criteria for reinforced concrete columns for nonlinear procedures (ASCE 41-13, Table 10-8)

Conditions			Modeling Parameters[a]: Plastic Rotations Angle (radians) a	Modeling Parameters[a]: Plastic Rotations Angle (radians) b	Modeling Parameters[a]: Residual Strength Ratio c	Acceptance Criteria[a]: Plastic Rotations Angle (radians), Performance Level IO	LS	CP
Condition i.[b]								
$\frac{P}{A_g f'_c}$ [c]	$\rho = \frac{A_v}{b_w s}$							
≤0.1	≥0.006		0.035	0.060	0.2	0.005	0.045	0.060
≥0.6	≥0.006		0.010	0.010	0.0	0.003	0.009	0.010
≤0.1	=0.002		0.027	0.034	0.2	0.005	0.027	0.034
≥0.6	=0.002		0.005	0.005	0.0	0.002	0.004	0.005
Condition ii.[b]								
$\frac{P}{A_g f'_c}$ [c]	$\rho = \frac{A_v}{b_w s}$	$\frac{V}{b_w d \sqrt{f'_c}}$ [d]						
≤0.1	≥0.006	≤3 (0.25)	0.032	0.060	0.2	0.005	0.045	0.060
≤0.1	≥0.006	≥6 (0.5)	0.025	0.060	0.2	0.005	0.045	0.060
≥0.6	≥0.006	≤3 (0.25)	0.010	0.010	0.0	0.003	0.009	0.010
≥0.6	≥0.006	≥6 (0.5)	0.008	0.008	0.0	0.003	0.007	0.008
≤0.1	≤0.0005	≤3 (0.25)	0.012	0.012	0.2	0.005	0.010	0.012
≤0.1	≤0.0005	≥6 (0.5)	0.006	0.006	0.2	0.004	0.005	0.006
≥0.6	≤0.0005	≤3 (0.25)	0.004	0.004	0.0	0.002	0.003	0.004
≥0.6	≤0.0005	≥6 (0.5)	0.0	0.0	0.0	0.0	0.0	0.0
Condition iii.[b]								
$\frac{P}{A_g f'_c}$ [c]	$\rho = \frac{A_v}{b_w s}$							
≤0.1	≥0.006		0.0	0.060	0.0	0.0	0.045	0.060
≥0.6	≥0.006		0.0	0.008	0.0	0.0	0.007	0.008
≤0.1	≤0.0005		0.0	0.006	0.0	0.0	0.005	0.006
≥0.6	≤0.0005		0.0	0.0	0.0	0.0	0.0	0.0
Condition iv. Columns controlled by inadequate development or splicing along the clear height[b]								
$\frac{P}{A_g f'_c}$ [c]	$\rho = \frac{A_v}{b_w s}$							
≤0.1	≥0.006		0.0	0.060	0.4	0.0	0.045	0.060
≥0.6	≥0.006		0.0	0.008	0.4	0.0	0.007	0.008
≤0.1	≤0.0005		0.0	0.006	0.2	0.0	0.005	0.006
≥0.6	≤0.0005		0.0	0.0	0.0	0.0	0.0	0.0

NOTE: f'_c is in lb/in.^{2} (MPa) units.
[a]Values between those listed in the table should be determined by linear interpolation.
[b]Refer to Section 10.4.2.2.2 for definition of conditions i, ii, and iii. Columns are considered to be controlled by inadequate development or splices where the calculated steel stress at the splice exceeds the steel stress specified by Eq. (10-2). Where more than one of conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.
[c]Where $P > 0.7 A_g f'_c$, the plastic rotation angles should be taken as zero for all performance levels unless the column has transverse reinforcement consisting of hoops with 135-degree hooks spaced at $\leq d/3$ and the strength provided by the hoops (V_s) is at least 3/4 of the design shear. Axial load P should be based on the maximum expected axial loads caused by gravity and earthquake loads.
[d]V is the design shear force from NSP or NDP.

Table 3- 39: Transverse reinforcement details: condition to be used for columns in ASCE 41-13 Table 10-8 (ASCE 41-13, Table 10-11)

Shear Capacity Ratio	ACI 318 Conforming Seismic Details with 135-Degree Hooks	Closed Hoops with 90-Degree Hooks	Other (Including Lap-Spliced Transverse Reinforcement)
$V_p/V_o \leq 0.6$	i[a]	ii	ii
$1.0 \geq V_p/V_o > 0.6$	ii	ii	iii
$V_p/V_o > 1.0$	iii	iii	iii

[a]To qualify for condition i, a column should have $A_v/b_w s \geq 0.002$ and $s/d \leq 0.5$ within flexural plastic hinge region. Otherwise, the column is assigned to condition ii.

1. Condition i: Flexure failure;
2. Condition ii: Flexure-shear failure, where yielding in flexure is expected before shear failure; and
3. Condition iii: Shear failure.

Table 3- 40: Database results for modelling parameters in ASCE 41-13 Table 10-8 (ASCE 41-13, Table C10-1)

Modeling Parameter	No. of Tests	Mean ($\theta_{pmeas}/\theta_{ptable}$)	$\beta(\theta_{pmeas}/\theta_{ptable})$	Probability of Failure[a]
a for condition i	141	1.44	0.50	30%
a for condition ii	31	2.23	0.47	6%
a for condition iii	34	4.66	0.48	0.1%
b for condition ii	28	1.97	0.50	13%

[a]Assuming a lognormal distribution for ($\theta_{pmeas}/\theta_{pcalc}$).

Table 3- 41: Numerical acceptance criteria for reinforced concrete columns linear procedures (ASCE 41-13, Table 10-9)

			m-Factors[a]				
			Performance Level				
				Component Type			
				Primary		Secondary	
Conditions			IO	LS	CP	LS	CP
Condition i[b]							
$\frac{P}{A_g f_c'}$ [c]	$\rho=\frac{A_v}{b_w s}$						
≤0.1	≥0.006		2	2.5	3	4	5
≥0.6	≥0.006		1.25	1.8	1.9	1.9	2
≤0.1	≤0.002		2	2	2.6	2.6	3
≥0.6	≤0.002		1.1	1.1	1.2	1.2	1.4
Condition ii[b]							
$\frac{P}{A_g f_c'}$ [c]	$\rho=\frac{A_v}{b_w s}$	$\frac{V}{b_w d\sqrt{f_c'}}$ [d]					
≤0.1	≥0.006	≤3 (0.25)	2	2.5	3	4	5
≤0.1	≥0.006	≥6 (0.5)	2	2	2.5	4	5
≥0.6	≥0.006	≤3 (0.25)	1.25	1.8	1.9	1.9	2
≥0.6	≥0.006	≥6 (0.5)	1.25	1.5	1.6	1.6	1.8
≤0.1	≤0.0005	≤3 (0.25)	1.2	1.3	1.4	1.4	1.6
≤0.1	≤0.0005	≥6 (0.5)	1	1	1.1	1.1	1.2
≥0.6	≤0.0005	≤3 (0.25)	1	1	1.1	1.1	1.2
≥0.6	≤0.0005	≥6 (0.5)	1	1	1	1	1
Condition iii[b]							
$\frac{P}{A_g f_c'}$ [c]	$\rho=\frac{A_v}{b_w s}$						
≤0.1	≥0.006		1	1	1	4	5
≥0.6	≥0.006		1	1	1	1.6	1.8
≤0.1	≤0.002		1	1	1	1.1	1.2
≥0.6	≤0.002		1	1	1	1	1
Condition iv. Columns controlled by inadequate development or splicing along the clear height[b]							
$\frac{P}{A_g f_c'}$ [c]	$\rho=\frac{A_v}{b_w s}$						
≤0.1	≥0.006		1	1	1	4	5
≥0.6	≥0.006		1	1	1	1.6	1.8
≤0.1	≤0.002		1	1	1	1.1	1.2
≥0.6	≤0.002		1	1	1	1	1

NOTE: f_c' is in lb/in.^{2} (MPa) units.
[a]Values between those listed in the table should be determined by linear interpolation.
[b]Refer to Section 10.4.2.2.2 for definition of conditions i, ii, and iii. Columns are considered to be controlled by inadequate development or splices where the calculated steel stress at the splice exceeds the steel stress specified by Eq. (10-2). Where more than one of conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.
[c]Where $P > 0.7A_g f_c'$, the *m*-factor should be taken as unity for all performance levels unless the column has transverse reinforcement consisting of hoops with 135-degree hooks spaced at ≤ $d/3$ and the strength provided by the hoops (V_s) is at least 3/4 of the design shear. P is the design axial force in the member. Alternatively, axial loads determined based on a limit-state analysis can be used.
[d]V is the design shear force calculated using limit-state analysis procedures in accordance with Section 10.4.2.4.1.

3.3.4.3. Joints

Table 3- 42 provide a summary of procedures to determine beam-column joint shear strength from the four codified assessment procedures.

In NZSEE 2006, the determination of joint shear strength is referred to design code SNZ 1995 (i.e. NZS3101:1995), taking degradation of shear strength due to cyclic loading into consideration. It is worth noting that the guidelines are only applicable for joint without shear reinforcement in the joint region. Similar to the guidelines provided in NZSEE 2006, in EN 1998-3:2005 and NTC 2008, the determination of joint shear strength is referred to European design standards. In ASCE 41-13, similar to beam and column force-deformation models, joint models are adopted. In Table 3- 43 and Table 3-45, information regarding modelling parameters and numerical acceptance criteria applied in nonlinear and linear analysis of joints is gathered. The details associated with the analysis approaches are presented in Chapter 4.

Table 3- 42: Determination of joint shear strength/capacity

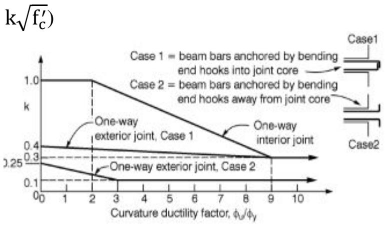
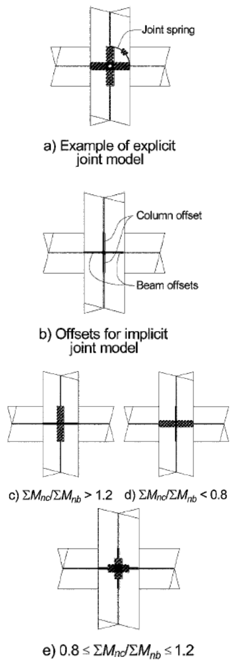
Joint	NZSEE 2006	ASCE 41-13	EN 1998-3:2005	NTC 2008
Strengths/ Capacities	**Shear strength:** (for interior and exterior beam-column joints without shear reinforcement, the probable horizontal joint shear force that can be resisted as:) $V_{pjh} = 0.85 v_{ch} b_j h$ $= 0.85k\sqrt{f'_c}\sqrt{1+\frac{N^*}{A_g k\sqrt{f'_c}}}b_j h$ $\le 1.92\sqrt{f'_c}b_j h$ Where: Vijh = probable horizontal joint shear force v_{ch} = nominal horizontal joint shear stress carried by a diagonal compressive strut mechanism crossing joint k = 1.0 for interior joint k = 0.4 for exterior joint with beam longitudinal bars anchored by bending the hooks into the joint core k = 0.25 for exterior joint with beam longitudinal bars anchored by bending the hooks away from the joint core (into column above or below) fc' = expected concrete compressive strength bj = effective width of the joint (normally the column width) h = depth of column Ag = area of joint, Ag=bjhc NOTE: (1) Conservative approach, particularly if there are no plastic hinges undergoing cyclic deformations in the post-elastic range adjacent to the joint core. (2) Recommended values for k: based on the estimated maximum nominal horizontal joint core shear stress, calculated in the conventional way, resisted by beam-column joints in tests without joint shear reinforcement and without axial load. The term indicating the influence of axial load, $1+\frac{N^*}{A_g k\sqrt{f'_c}}$ was obtained by assuming that the diagonal tensile strength of the concrete was $k\sqrt{f'_c}$ and calculating using Mohr's circle for stress the horizontal shear stress required to induce this diagonal (principal) tensile stress when the vertical compressive stress is N*/Ag (*Hakuto et al (2000))* Degradation of shear strength: (degradation of $k\sqrt{f'_c}$)  NOTE: Interior joints are not as vulnerable as exterior joint Exterior joints with the 90° hooks at the end of the longitudinal beam bars bent away from the joint core do not perform well because the beam bar hooks do not properly engage the corner to corner diagonal compression strut	**Stiffness:** Where not modelled explicitly, should be implicitly modelled by adjusting a centreline model:  Account for shear flexibility, stiffness values used for beams and columns, flexibility resulting from bar slip **Shear strength:** $V_n = \lambda\gamma\sqrt{f'_c}A_j$ (lb/in²) $V_n = 0.083\lambda\gamma\sqrt{f'_c}A_j$ (MPa) λ = 0.75 for lightweight aggregate concrete λ = 1.0 for normal weight aggregate concrete Aj = effective horizontal joint area with dimensions defined by: (a joint depth equal to the column dimension in the direction of framing and a joint width equal to the: min[column width, beam width plus joint depth, twice the smaller perpendicular distance from the longitudinal axis of the beam to the column side]) γ = Table 10-12 Table 10-10 and 10-14	**Shear strength:** NC limit state: the diagonal compression shall not exceed the compressive strength of a concrete in the presence of transverse tensile strains. This requirement can be satisfied by means of the subsequent rules: Interior joints: $V_{jhd} \le \eta f_{cd}\sqrt{1-\frac{\nu_d}{\eta}}b_j h_{jc}$ $V_{jhd} = \gamma_{Rd}(A_{s1}+A_{s2})f_{yd} - Vc$ $\eta = 0.6(1-\frac{f_{ck}}{250})$ h_{jc}: the distance between extreme layers of column reinforcement ν_d: the normalized axial force in the column above the joint Exterior joints: $V_{jhd} < 0.8\,\eta f_{cd}\sqrt{1-\frac{\nu_d}{\eta}}b_j h_{jc}$ $V_{jhd} = \gamma_{Rd}A_{s1}f_{yd} - Vc$ b_j=the effective joint width If $b_c > b_w$: $b_j = \min\{b_c; (b_w + 0.5h_c)\}$ If $b_c < b_w$: $b_j = \min\{b_w; (b_c + 0.5h_c)\}$ SD and DL limit states: The verification is not required unless these LS are the only ones to be checked (in that case see NC)	**Diagonal tensile/compressive strength:** Verification of strength must be done only for not entirely confined beam- column joints: For diagonal tensile strength: $\sigma_{nt} = \left\lvert\frac{N}{2A_g} - \sqrt{\left(\frac{N}{2A_g}\right)^2+\left(\frac{V_n}{A_g}\right)^2}\right\rvert$ and $\sigma_{nt} \le 0.3\sqrt{f_c}$ (f_c in MPa) For diagonal compressive strength: $\sigma_{nc} = \frac{N}{2A_g} + \sqrt{(\frac{N}{2A_g})^2+(\frac{V_n}{A_g})^2} \le 0.5f_c$ N=axial load on the upper column Vn=total shear in the joint considering the shear from the upper column and the shear related to tensile stress of superior longitudinal bars of beam Ag=horizontal section of joint Material strength are average from in situ tests and other information divided by CF and partial safety factor

Table 3- 43: Modelling parameters and numerical acceptance criteria for reinforced concrete beam-column joints for nonlinear procedures (ASCE 41-13 Table 10-10)

Conditions			Modeling Parameters[a]			Acceptance Criteria[a]		
			Plastic Rotations Angle (radians)		Residual Strength Ratio	Plastic Rotations Angle (radians)		
						Performance Level		
			a	b	c	IO	LS	CP
Condition i. Interior joints (Note: For classification of joints, refer to Fig. 10-3)								
$\frac{P}{A_g f'_c}$ [b]	Transverse reinforcement[c]	$\frac{V}{V_n}$ [d]						
≤0.1	C	≤1.2	0.015	0.03	0.2	0.0	0.02	0.03
≤0.1	C	≥1.5	0.015	0.03	0.2	0.0	0.015	0.02
≥0.4	C	≤1.2	0.015	0.025	0.2	0.0	0.015	0.025
≥0.4	C	≥1.5	0.015	0.2	0.2	0.0	0.015	0.02
≤0.1	NC	≤1.2	0.005	0.2	0.2	0.0	0.015	0.02
≤0.1	NC	≥1.5	0.005	0.015	0.2	0.0	0.01	0.015
≥0.4	NC	≤1.2	0.005	0.015	0.2	0.0	0.01	0.015
≥0.4	NC	≥1.5	0.005	0.015	0.2	0.0	0.01	0.015
Condition ii. Other joints (Note: For classification for joints, refer to Fig. 10-3)								
$\frac{P}{A_g f'_c}$ [b]	Transverse reinforcement[c]	$\frac{V}{V_n}$ [d]						
≤0.1	C	≤1.2	0.01	0.02	0.2	0.0	0.015	0.02
≤0.1	C	≥1.5	0.01	0.015	0.2	0.0	0.01	0.015
≥0.4	C	≤1.2	0.01	0.02	0.2	0.0	0.015	0.02
≥0.4	C	≥1.5	0.01	0.015	0.2	0.0	0.01	0.015
≤0.1	NC	≤1.2	0.005	0.01	0.2	0.0	0.0075	0.01
≤0.1	NC	≥1.5	0.005	0.01	0.2	0.0	0.0075	0.01
≥0.4	NC	≤1.2	0.0	0.0075	0.0	0.0	0.005	0.0075
≥0.4	NC	≥1.5	0.0	0.0075	0.0	0.0	0.005	0.0075

[a]Values between those listed in the table should be determined by linear interpolation.
[b]P is the design axial force on the column above the joint calculated using limit-state analysis procedures in accordance with Section 10.4.2.4, and A_g is the gross cross-sectional area of the joint.
[c]"C" and "NC" are abbreviations for conforming and nonconforming transverse reinforcement. Joint transverse reinforcement is conforming if hoops are spaced at ≤ $h_c/2$ within the joint. Otherwise, the transverse reinforcement is considered nonconforming.
[d]V is the design shear force from NSP or NDP, and V_n is the shear strength for the joint. The shear strength should be calculated according to Section 10.4.2.3.

Table 3- 44: Values of γ for joint strength calculation (ASCE 41-13, Table 10-12)

Transverse Reinforcement[b]	Value of γ				
	Condition i: Interior Joints[a]		Condition ii: Other Joints		
	Interior Joint with Transverse Beams	Interior Joint without Transverse Beams	Exterior Joint with Transverse Beams	Exterior Joint without Transverse Beams	Knee Joint with or without Transverse Beams
C	20	15	15	12	8
NC	12	10	8	6	4

[a]For classification of joints, refer to Fig. 10-3.
[b]"C" and "NC" are abbreviations for conforming and nonconforming transverse reinforcement. Joint transverse reinforcement is conforming if hoops are spaced at ≤ $h_c/2$ within the joint. Otherwise, the transverse reinforcement is considered nonconforming.

Table 3- 45: Numerical acceptance criteria for reinforced concrete beam-column joints for linear procedures (ASCE 41-13, Table 10-14)

Conditions			m-Factors[a]				
			Performance Level				
				Component Type			
				Primary		Secondary	
			IO	LS	CP	LS	CP
Condition i. Interior joints (for classification of joints, refer to Fig. 10-3)							
$\frac{P}{A_g f'_c}$ [b]	Transverse reinforcement[c]	$\frac{V}{V_n}$ [d]					
≤0.1	C	≤1.2	1	1	1	3	4
≤0.1	C	≥1.5	1	1	1	2	3
≥0.4	C	≤1.2	1	1	1	3	4
≥0.4	C	≥1.5	1	1	1	2	3
≤0.1	NC	≤1.2	1	1	1	2	3
≤0.1	NC	≥1.5	1	1	1	2	3
≥0.4	NC	≤1.2	1	1	1	2	3
≥0.4	NC	≥1.5	1	1	1	2	3
Condition ii. Other joints (for classification of joints, refer to Fig. 10-3)							
$\frac{P}{A_g f'_c}$ [b]	Transverse reinforcement[c]	$\frac{V}{V_n}$ [d]					
≤0.1	C	≤1.2	1	1	1	3	4
≤0.1	C	≥1.5	1	1	1	2	3
≥0.4	C	≤1.2	1	1	1	3	4
≥0.4	C	≥1.5	1	1	1	2	3
≤0.1	NC	≤1.2	1	1	1	2	3
≤0.1	NC	≥1.5	1	1	1	2	3
≥0.4	NC	≤1.2	1	1	1	1.5	2
≥0.4	NC	≥1.5	1	1	1	1.5	2

[a]Values between those listed in the table should be determined by linear interpolation.
[b]P is the design axial force on the column above the joint calculated using limit-state analysis procedures in accordance with Section 10.4.2.4. A_g is the gross cross-sectional area of the joint.
[c]V is the design shear force and V_n is the shear strength for the joint. The design shear force and shear strength should be calculated according to Section 10.4.2.4.1 and Section 10.4.2.3, respectively.
[d]"C" and "NC" are abbreviations for conforming and nonconforming transverse reinforcement, respectively. Transverse reinforcement is conforming if hoops are spaced at ≤ $h_c/2$ within the joint. Otherwise, the transverse reinforcement is considered nonconforming.

3.3.4.4. Walls (requires future research and investigation)

Table 3- 46 provides a summary of procedures to determine wall strength from the four codified assessment procedures.

Table 3- 46: Determination of wall strengths/capacities

Wall	NZSEE 2006	ASCE 41-13	EN 1998-3:2005	NTC 2008
Strengths/ Capacities	**Flexural strength:** Centre of mass for the building Effective stiffness of each wall (NZS3101: Part 2:1995 Table C3.1) Probable flexural capacity for each structural wall (at wall base) **Shear strength:** (Probable shear) Concrete contribution to shear capacity: $v_{cp} = 0.6\sqrt{\frac{f'_c}{25}\frac{N^*}{A_g}}$ (The equation was based on concrete compression strength of 25MPa, in order to allow some benefit to be derived when the assessed concrete in the existing structure is stronger.) $v_{cp} = \frac{(5-\mu_{sd})\left(\sqrt{f'_c}+\frac{N^*}{A_g}\right)}{16}$ (The equation is applicable to elements of limited ductility (i.e. for moderate displacement ductility demand, say, less than 3). Higher nominal shear stresses were obtained using this equation.) $V_c = v_{cp}b_w d$ Reinforcing steel contribution: $V_s = \frac{A_v f_{yt} d}{s}$ **Total probable lateral force :** $V_{prob} = \frac{1.5\sum M_{wp}}{h_w}$ **Eccentricity:** Centre of resistance CV: $\bar{y}_{CV} = \frac{\sum_1^i V_{wxpi}y_i}{\sum_1^i V_{wxpi}}$? $\bar{x}_{CV} = \frac{\sum_1^i V_{wypi}x_i}{\sum_1^i V_{wypi}}$? Or: $\bar{y}_{CV} = \frac{\sum_1^i M_{wxpi}y_i}{\sum_1^i M_{wxpi}}$? $\bar{x}_{CV} = \frac{\sum_1^i M_{wypi}x_i}{\sum_1^i M_{wypi}}$? Centre of mass CM: $\bar{y}_{CM} = \frac{\sum_1^i A_{wxi}y_i}{\sum_1^i A_{wxi}}$? $\bar{x}_{CM} = \frac{\sum_1^i A_{wyi}x_i}{\sum_1^i A_{wyi}}$? Or: (if all walls have same/similar thickness) $\bar{y}_{CM} = \frac{\sum_1^i L_{wxi}y_i}{\sum_1^i L_{wxi}}$? $\bar{x}_{CM} = \frac{\sum_1^i L_{wyi}x_i}{\sum_1^i L_{wyi}}$? $e_{vy} = \bar{y}_{CV} - \bar{y}_{CM}$, $e_{vx} = \bar{x}_{CV} - \bar{x}_{CM}$ **Curvature ductility capacity:** For walls with no confinement: $\mu_\varphi = \frac{1.25}{\frac{c}{L_w}}$, and $A_r = \frac{h_{eff}}{L_w}$ (Required curvature ductility capacity of cantilever wall sections as a function of displacement ductility demand and aspect ratio) $h_{eff} = 0.67h_w$ for cantilever walls $h_{eff} = 0.5h_w$ for dual systems $L_p = 0.08H_w + 0.022d_b f_y$ $\mu_d = (\mu_\varphi - 1)\times 3\left(\frac{L_p}{H_w}\right)\left[1-0.5\left(\frac{L_p}{H_w}\right)\right]+1$ (Using this way, the calculated displacement ductility capacities are CLOSE to the results obtained from the Figure) For walls with confinement: $\mu = 40\left[\frac{\frac{A_{sh}}{S_h h''}}{\frac{A_g^* f_c}{A_c^* f_{yh}}\left(\frac{c}{L_w}-0.07\right)} - 0.1\right]$ (NZSEE 2006Eq7(36) is incorrect) **Deformation capacity:** $\varphi_{wy} = \frac{1.8\varepsilon_y}{L_w}$ $\mu_{wc} = 0.025(A_{re}-0.25)\frac{L_w}{U_{wy}} + 1 = \frac{0.04(A_{re}-0.25)}{\varepsilon_y {A_{re}}^2} + 1$ $U_{wy} = \left(\frac{1}{2}\varphi_{wy}h_{eff}\right)\times\left(\frac{2}{3}h_{eff}\right) = \frac{1}{3}\varphi_{wy}h_{eff}^2 = 0.6\varepsilon_y A_{re} h_{eff}$ $U_{wp} = U_{wy}(\mu_{wc}-1)$, $U_{wc} = U_{wy} + U_{wp} = U_{wy}\times\mu_{wc}$ Displacement capacity of the building is controlled by the wall element with the smallest A_{re}. Thus, the equations need to be considered only for the wall of the system with the greatest length. $\delta_{wy} = \frac{1}{2}\varphi_{wy}h_{eff} = \frac{U_{wy}}{\frac{2}{3}h_{eff}} = 0.9\varepsilon_y A_{re}$ $\delta_{wp} = \frac{U_{wp}}{h_{eff}-\frac{1}{2}L_p}$, $\delta_{w,max} = \delta_{wy} + \delta_{wp} \leq 2.5\%$ It is assumed that the stiffness of an element is proportional to its probable strength. Each element can be expected to enter the inelastic domain at a different lateral displacement, and the superposition of element responses leads to the non-linear total response of the system. Although a system does not have a distinct nominal yield displacement, a reference system nominal yield displacement is defined as: $U_{sy} = \frac{\sum M_{wp}}{h_{eff}\sum k_w}$, $U_s = U_{wc,min}$, $\mu_s = \frac{U_s}{U_{sy}} = \frac{U_{wc,min}}{U_{sy}}$	**Stiffness:** Based on the material properties, component dimensions, reinforcement quantities, boundary conditions, and current state of the member with respect to cracking and stress levels. LSP, LDP, NSP, NDP Table 10-5 **Strength:** (Determined considering the potential for failure in flexure, shear, or development under combined gravity and lateral load.) Flexural strength: ACI 318 Chapter 10, Table 10-19 Shear strength: ACI 318 Chapter 21, Table 10-20	**Ductile mechanism – under flexure:** NC limit state: Same as beams and columns SD limit state: Same as beams and columns DL limit state: chord rotation capacity at yielding for walls of rectangular, T- or barbelled section $\theta_y = \phi_y\frac{L_v+a_V z}{3} + 0.002\left(1-\frac{0.135L_v}{h}\right) + \frac{\varepsilon_y d_b f_y}{(d-d')\cdot 6\sqrt{f_c}}$ $a_V z$ = the tension shift of the bending moment diagram ($a_V = 1$ if $M_y > L_v V_{R,c}$ otherwise $a_V = 0$ if $M_y < L_v V_{R,c}$) Or from the alternative expressions: $\theta_y = \phi_y\frac{L_v+a_V z}{3} + 0.002\left(1-\frac{0.125L_v}{h}\right) + \frac{0.13\phi_y d_b f_y}{\sqrt{f_c}}$ **Brittle mechanism – shear:** NC limit state: Same as for beams and columns Add: the shear strength of a concrete wall, V_R, may not be taken greater than the value corresponding to failure by web crushing, $V_{R,max}$, which under cyclic loading may be calculated as: $V_{R,max} = \frac{0.85\left(1-0.06\min\left(5;\mu_\Delta^{pi}\right)\right)}{\gamma_{el}}\left(1 + 1.8\min\left(0.15;\frac{N}{A_c f_c}\right)\right)(1 + 0.25\max(1.75;100\rho_{tot}) - 0.2\min(2;\frac{L_v}{h})\sqrt{f_c}b_w z$ The minimum of shear resistance calculated in accordance with EN1992-1-1:2004 or by means of the previous expressions should be used in the assessment SD and DL limit states: The verification is not required unless these LS are the only ones to be checked (in that case see NC)	**Ductile mechanism – under flexure:** Collapse limit state: Same as beams and columns Life safeguard limit state: Same as beams and columns Serviceability limit state: chord rotation capacity at yielding for walls: $\theta_y = \frac{\phi_y L_v}{3} + 0.002\left(1-\frac{0.125L_v}{h}\right) + \frac{0.13\phi_y d_b f_y}{\sqrt{f_c}}$ f_c and f_y are obtained as average from in situ tests divided by partial safety factor and CF

Table 3- 47: Modelling parameters and numerical acceptance criteria for RC shear walls and associated components controlled by flexure for nonlinear procedures (ASCE 41-13, Table 10-19)

Conditions			Plastic Hinge Rotation (radians) a	b	Residual Strength Ratio c	Acceptable Plastic Hinge Rotation[a] (radians) Performance Level IO	LS	CP
i. Shear walls and wall segments								
$\frac{(A_s - A_s')f_y + P}{t_w l_w f_c'}$	$\frac{V}{t_w l_w \sqrt{f_c'}}$	Confined Boundary[b]	0.015					
≤0.1	≤4	Yes	0.010	0.020	0.75	0.005	0.015	0.020
≤0.1	≥6	Yes	0.009	0.015	0.40	0.004	0.010	0.015
≥0.25	≤4	Yes	0.005	0.012	0.60	0.003	0.009	0.012
≥0.25	≥6	Yes	0.008	0.010	0.30	0.0015	0.005	0.010
≤0.1	≤4	No	0.006	0.015	0.60	0.002	0.008	0.015
≤0.1	≥6	No	0.003	0.010	0.30	0.002	0.006	0.010
≥0.25	≤4	No	0.002	0.005	0.25	0.001	0.003	0.005
≥0.25	≥6	No	0.002	0.004	0.20	0.001	0.002	0.004
ii. Shear wall coupling beams[c]								
Longitudinal reinforcement and transverse reinforcement[d]		$\frac{V}{t_w l_w \sqrt{f_c'}}$		0.050				
Conventional longitudinal reinforcement with conforming transverse reinforcement		≤3	0.025	0.040	0.75	0.010	0.025	0.050
		≥6	0.020	0.035	0.50	0.005	0.020	0.040
Conventional longitudinal reinforcement with nonconforming transverse reinforcement		≤3	0.020	0.025	0.50	0.006	0.020	0.035
		≥6	0.010	0.050	0.25	0.005	0.010	0.025
Diagonal reinforcement		NA	0.030	0.050	0.80	0.006	0.030	0.050

[a]Linear interpolation between values listed in the table shall be permitted.
[b]A boundary element shall be considered confined where transverse reinforcement exceeds 75% of the requirements given in ACI 318 and spacing of transverse reinforcement does not exceed $8d_b$. It shall be permitted to take modeling parameters and acceptance criteria as 80% of confined values where boundary elements have at least 50% of the requirements given in ACI 318 and spacing of transverse reinforcement does not exceed $8d_b$. Otherwise, boundary elements shall be considered not confined.
[c]For coupling beams spanning <8 ft 0 in., with bottom reinforcement continuous into the supporting walls, acceptance criteria values shall be permitted to be doubled for LS and CP performance.
[d]Conventional longitudinal reinforcement consists of top and bottom steel parallel to the longitudinal axis of the coupling beam. Conforming transverse reinforcement consists of (a) closed stirrups over the entire length of the coupling beam at a spacing ≤ $d/3$, and (b) strength of closed stirrups $V_s \geq 3/4$ of required shear strength of the coupling beam.

Table 3- 48: Modelling parameters and numerical acceptance criteria for RC shear walls and associated components controlled by shear for nonlinear procedures (ASCE 41-13, Table 10-21)

Conditions		Total Drift Ratio (%), or Chord Rotation (radians)[a] d	e	g	Strength Ratio c	f	Acceptable Total Drift (%) or Chord Rotation (radians)[a] Performance Level IO	LS	CP
i. Shear walls and wall segments[b]									
$\frac{(A_s - A_s')f_y + P}{t_w l_w f_c'} \leq 0.05$		1.0	2.0	0.4	0.20	0.6	0.40	1.5	2.0
$\frac{(A_s - A_s')f_y + P}{t_w l_w f_c'} > 0.05$		0.75	1.0	0.4	0.0	0.6	0.40	0.75	1.0
ii. Shear wall coupling beams[c]									
Longitudinal reinforcement and transverse reinforcement[d]	$\frac{V}{t_w l_w \sqrt{f_c'}}$								
Conventional longitudinal reinforcement with conforming transverse reinforcement	≤3	0.02	0.030		0.60		0.006	0.020	0.030
	≥6	0.016	0.024		0.30		0.005	0.016	0.024
Conventional longitudinal reinforcement with nonconforming transverse reinforcement	≤3	0.012	0.025		0.40		0.006	0.010	0.020
	≥6	0.008	0.014		0.20		0.004	0.007	0.012

[a]For shear walls and wall segments, use drift; for coupling beams, use chord rotation; refer to Figures 10-5 and 10-6.
[b]For shear walls and wall segments where inelastic behavior is governed by shear, the axial load on the member must be ≤ $0.15A_g f_c'$; otherwise, the member must be treated as a force-controlled component.
[d]Conventional longitudinal reinforcement consists of top and bottom steel parallel to the longitudinal axis of the coupling beam. Conforming transverse reinforcement consists of (a) closed stirrups over the entire length of the coupling beam at a spacing ≤ $d/3$ and (b) strength of closed stirrups $V_s \geq 3/4$ of required shear strength of the coupling beam.
[c]For coupling beams spanning <8 ft 0 in., with bottom reinforcement continuous into the supporting walls, acceptance criteria values shall be permitted to be doubled for LS and CP performance.

Table 3- 49: Reinforced concrete shear wall component types (ASCE 41-13, Table C10-2)

Component Type per FEMA 306 (1998b)		Description	ASCE 41 Designation
RC1	Isolated wall or stronger wall pier	Stronger than beam or spandrel components that may frame into it so that nonlinear behavior (and damage) is generally concentrated at the base, with a flexural plastic hinge or shear failure. Includes isolated (cantilever) walls. If the component has a major setback or cutoff of reinforcement above the base, this section should be also checked for nonlinear behavior.	Monolithic reinforced concrete wall or vertical wall segment
RC2	Weaker wall pier	Weaker than the spandrels to which it connects; characterized by flexural hinging top and bottom or shear failure.	
RC3	Weaker spandrel or coupling beam	Weaker than the wall piers to which it connects; characterized by hinging at each end, shear failure, or sliding shear failure.	Horizontal wall segment or coupling beam
RC4	Stronger spandrel	Should not suffer damage because it is stronger than attached wall piers. If this component is damaged, it should probably be reclassified as RC3.	
RC5	Pier-spandrel panel zone	Typically not a critical area in RC walls.	Wall segment

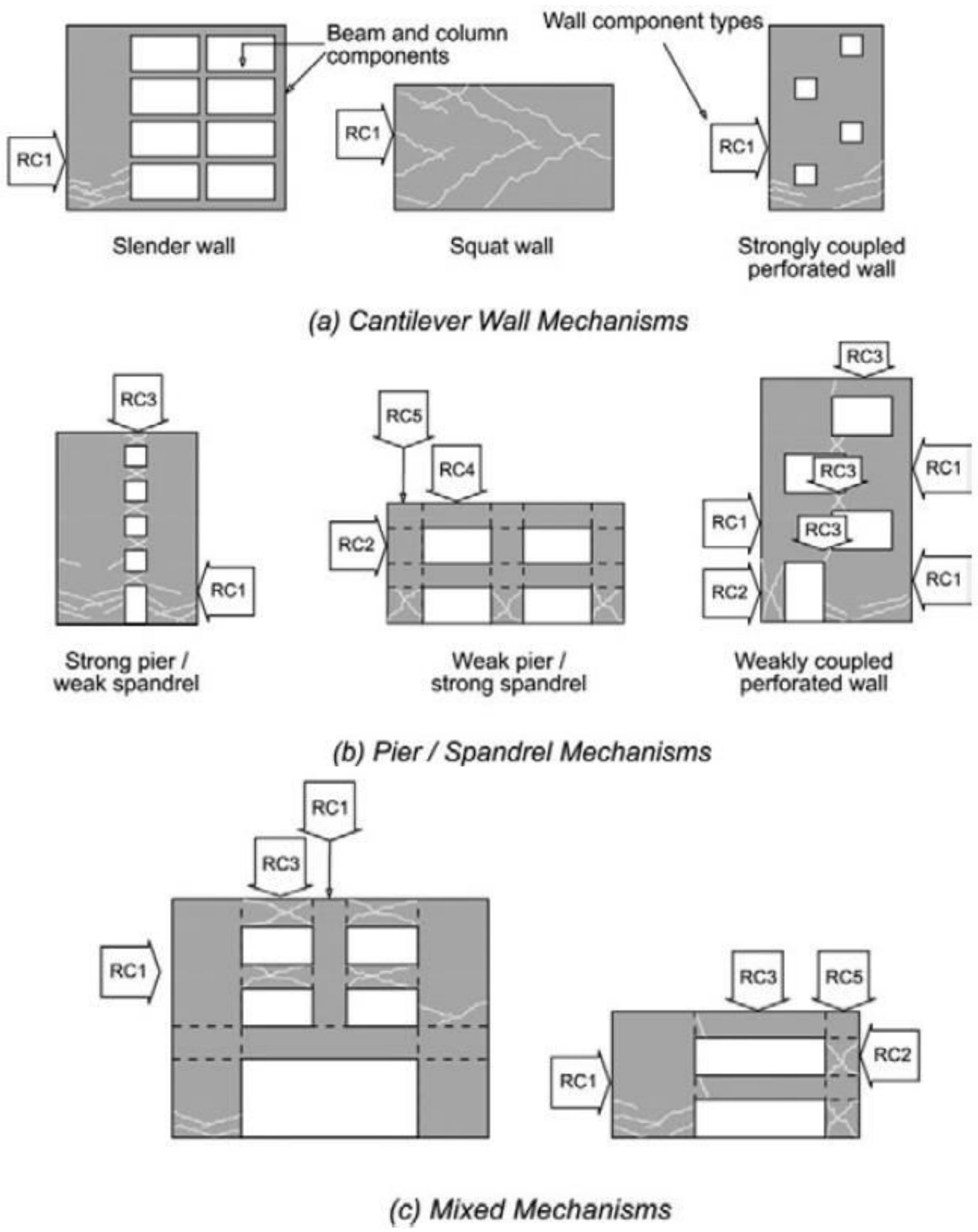


Figure 3- 22: Identification of component types in the concrete shear wall elements (FEMA 306 (1998b))

Table 3- 50: Numerical acceptance criteria for RC shear walls and associated components controlled by flexure for linear procedures (ASCE 41-13, Table 10-21)

Conditions			m-Factors[a]				
			Performance Level				
				Component Type			
				Primary		Secondary	
			IO	LS	CP	LS	CP
i. Shear walls and wall segments							
$\frac{(A_s - A'_s)f_y + P}{t_w l_w f'_c}$ [b]	$\frac{V}{t_w l_w \sqrt{f'_c}}$ [c]	Confined Boundary[d]					
≤0.1	≤4	Yes	2	4	6	6	8
≤0.1	≥6	Yes	2	3	4	4	6
≥0.25	≤4	Yes	1.5	3	4	4	6
≥0.25	≥6	Yes	1.25	2	2.5	2.5	4
≤0.1	≤4	No	2	2.5	4	4	6
≤0.1	≥6	No	1.5	2	2.5	2.5	4
≥0.25	≤4	No	1.25	1.5	2	2	3
≥0.25	≥6	No	1.25	1.5	1.75	1.75	2
ii. Shear wall coupling beams[e]							
Longitudinal reinforcement and transverse reinforcement[f]		$\frac{V}{t_w l_w \sqrt{f'_c}}$ [c]					
Conventional longitudinal reinforcement with conforming transverse reinforcement		≤3	2	4	6	6	9
		≥6	1.5	3	4	4	7
Conventional longitudinal reinforcement with nonconforming transverse reinforcement		≤3	1.5	3.5	5	5	8
		≥6	1.2	1.8	2.5	2.5	4
Diagonal reinforcement		NA	2	5	7	7	10

[a]Linear interpolation between values listed in the table shall be permitted.
[b]P is the design axial force in the member. Alternatively, use of axial loads determined based on a limit state analysis shall be permitted.
[c]V is the design shear force calculated using limit-state analysis procedures in accordance with Section 10.7.2.4.
[d]A boundary element shall be considered confined where transverse reinforcement exceeds 75% of the requirements given in ACI 318 and spacing of transverse reinforcement does not exceed $8d_b$. It shall be permitted to take modeling parameters and acceptance criteria as 80% of confined values where boundary elements have at least 50% of the requirements given in ACI 318 and spacing of transverse reinforcement does not exceed $8d_b$. Otherwise, boundary elements shall be considered not confined.
[e]For secondary coupling beams spanning <8 ft 0 in., with bottom reinforcement continuous into the supporting walls, secondary values shall be permitted to be doubled.
[f]Conventional longitudinal reinforcement consists of top and bottom steel parallel to the longitudinal axis of the coupling beam. Conforming transverse reinforcement consists of (a) closed stirrups over the entire length of the coupling beam at a spacing ≤ $d/3$, and (b) strength of closed stirrups $V_s \geq 3/4$ of required shear strength of the coupling beam.

Table 3- 51: Numerical acceptance criteria for RC shear walls and associated components controlled by shear for linear procedures (ASCE 41-13, Table 10-22)

Conditions		m-Factors				
		Performance Level				
			Component Type			
			Primary		Secondary	
		IO	LS	CP	LS	CP
i. Shear walls and wall segments[a]						
$\frac{(A_s - A'_s)f_y + P}{t_w l_w f'_c} \leq 0.05$		2	2.5	3	4.5	6
$\frac{(A_s - A'_s)f_y + P}{t_w l_w f'_c} > 0.05$		1.5	2	3	3	4
ii. Shear wall coupling beams[b]						
Longitudinal reinforcement and transverse reinforcement[c]	$\frac{V}{t_w l_w \sqrt{f'_c}}$ [d]					
Conventional longitudinal reinforcement with conforming transverse reinforcement	≤ 3	1.5	3	4	4	6
	≥ 6	1.2	2	2.5	2.5	3.5
Conventional longitudinal reinforcement with nonconforming transverse reinforcement	≤ 3	1.5	2.5	3	3	4
	≥ 6	1.2	1.2	1.5	1.5	2.5

[a]The shear shall be considered to be a force-controlled action for shear walls and wall segments where inelastic behavior is governed by shear and the design axial load is greater than $0.15A_g f'_c$. It shall be permitted to calculate the axial load based on a limit state analysis.
[b]For secondary coupling beams spanning <8'-0", with bottom reinforcement continuous into the supporting walls, secondary values shall be permitted to be doubled.
[c]Conventional longitudinal reinforcement consists of top and bottom steel parallel to the longitudinal axis of the coupling beam. Conforming transverse reinforcement consists of: (a) closed stirrups over the entire length of the coupling beam at a spacing ≤ $d/3$, and (b) strength of closed stirrups $V_s \geq 3/4$ of required shear strength of the coupling beam.
[d]V is the design shear force calculated using limit-state analysis procedures in accordance with Section 10.7.2.4.1.

3.3.5. Difference in the analysis approaches

Table 3- 52 provides a summary of analysis approaches adopted in the four codified assessment procedures. It can be concluded that similar analysis approaches are suggested in the different assessment procedures. At preliminary assessment level, the minimum level of analysis is required in all the four codified procedures. At the most detailed assessment level, ASCE 41-13, EN 1998-3:2005 and NTC 2008 suggested that analysis approach can be select from LSP, LDP, NSP and NDP depending on the objective of assessment, access of input information, availability of analysis tools, etc. Among all the analysis approaches presented in Table 3- 52, SLaMa is a unique analytical ("by-hand") method to predict nonlinear response. General reviews and critical comparisons among analysis approaches are shown in Chapter 4.

Table 3- 52: Analysis approaches adopted in the four codified assessment procedures

		Codified Seismic Assessment/Evaluation Procedures			
Level/Stage of Assessment/ Evaluation		**NZSEE 2006**	**ASCE 41-13**	**EN 1998-3: 2005**	**NTC 2008**
Screening with Minimal Level of Analysis Applied	Specific Title in Each Code/Guideline	Initial Seismic Assessment (ISA)	Tier 1 Screening Procedure	Evaluation of Knowledge Level	Evaluation of Knowledge Level
	Inputs and Requirements				
	Analysis Approch	**None**	**Simplified LSP**	**None**	**None**
	Evaluation Procedures and Applied Tools				
	Acceptance Criteria				
	Outputs				
	Limitations				
More Detailed Assessment than Screening	Specific Title in Each Code/Guideline	NONE	Tier 2 Deficiency-based Evaluation	Knowledge Level 1 Confidence Factor	Knowledge Level 1 Confidence Factor
	Inputs and Requirements				
	Analysis Approch		**LSP** **LDP**	**LSP** **LDP**	**LSP** **LDP**
	Evaluation Procedures and Applied Tools				
	Acceptance Criteria				
	Outputs				
	Limitations				
Detailed Assessment	Specific Title in Each Code/Guideline	Detailed Seismic Assessment (DSA)	Tier 3 Systematic Evaluation	Knowledge Level 2 Knowledge Level 3 Confidence Factor	Knowledge Level 2 Knowledge Level 3 Confidence Factor
	Inputs and Requirements				
	Analysis Approch	**LSP** **LDP** **SLaMa** **NSP** **NDP**	**LSP** **LDP** **NSP** **NDP**	**LSP** **LDP** **NSP** **NDP**	**LSP** **LDP** **NSP** **NDP**
	Evaluation Procedures and Applied Tools	General FB DB			
	Acceptance Criteria		LSPLDP NSPNDP		
	Outputs	LSP . LDP SLaMa NSP NDP	LSPLDP NSP NDP		
	Limitations				

3.4. Simplified Assessment Procedures

The assessment procedures can be simplified by either simplifying the evaluation process or simplifying the selected analysis approach.

Table 3- 53: Simplified procedures found in the four codified assessment procedures

Level/Stage of Assessment/ Evaluation		Codified Seismic Assessment/Evaluation Procedures: NZSEE 2006	ASCE 41-13	EN 1998-3: 2005	NTC 2008
Screening with Minimal Level of Analysis Applied	Specific Title in Each Code/Guideline	**Initial Seismic Assessment (ISA)**	**Tier 1 Screening Procedure**	**Evaluation of Knowledge Level**	**Evaluation of Knowledge Level**
	Inputs and Requirements				
	Analysis Approach				
	Evaluation Procedures and Applied Tools				
	Acceptance Criteria				
	Outputs				
	Limitations				
More Detailed Assessment than Screening	Specific Title in Each Code/Guideline	NONE	Tier 2 Deficiency-based Evaluation	**Knowledge Level 1 Confidence Factor**	**Knowledge Level 1 Confidence Factor**
	Inputs and Requirements				
	Analysis Approach		LSP LDP	LSP LDP	LSP LDP
	Evaluation Procedures and Applied Tools				
	Acceptance Criteria				
	Outputs				
	Limitations				
Detailed Assessment	Specific Title in Each Code/Guideline	Detailed Seismic Assessment (DSA)	Tier 3 Systematic Evaluation	Knowledge Level 2 Knowledge Level 3 Confidence Factor	Knowledge Level 2 Knowledge Level 3 Confidence Factor
	Inputs and Requirements				
	Analysis Approach	LSP LDP SLaMa NSP NDP	LSP LDP NSP NDP	LSP LDP NSP NDP	LSP LDP NSP NDP
	Evaluation Procedures and Applied Tools	General FB DB			
	Acceptance Criteria		LSPLDP NSPNDP		
	Outputs	LSP LDP **SLaMa** NSP NDP	LSPLDP NSP NDP		
	Limitations				

As shown in Table 3- 53, the highlighted procedures at the lowest assessment level, such as ISA (from NZSEE 2006), Tier 1 (from ASCE 41-13), KL1 (from EN 1998-3:2005 and NTC 2008), compared to the procedures at higher assessment levels, are much simpler, with less requirements of the data collection process, simpler estimation or calculation procedure, simpler definition of acceptance criteria, etc. However, such simplified assessment procedures can only provide preliminary results, and further evaluation processes are usually required to confirm the preliminary results. Hence, simplifying the evaluation process may not meet the objective of assessment to provide robust results.

Even though good results can be computed by conducting detailed assessment, the disadvantages should also be acknowledged, such as considerable amounts of time and research efforts to collect and compile data, great amounts of funds to back up the study and powerful computing or data analysing tools, complexity of the analysis approaches involved, etc. Under some circumstances, the accessibility of data, availability of researches (i.e. time, money, equipment, etc.) are limited to some extent that the comprehensive assessment may become impractical and unjustified (Pinho *et al.* 2002). Therefore, a simplified analysis adopted to reduce of the complexity and expenses without influentially reducing the robustness of the assessment outcomes is preferred.

Hence, in the thesis, the focus is casted on simplified analysis approaches to be adopted at detailed assessment level. As shown in Table 3- 52 or Table 3- 53, to assess a typical structure, four analyses can be selected – LSP, LDP, NSP, NDP, and as mentioned in Section 3.2.1.2.3 and Section 3.3.5, apart from the four analyses, NZSEE 2006 includes a unique SLaMA, which is an analytical ("by-hand") pushover analysis approach. Details of SLaMa and alternative simplified analysis approaches are shown in Chapter 4 and Chapter 5, and the evaluation of the capability of these approaches are presented from Chapter 7 to Chapter 9.

It is worth noticing that the simplified NSP (Nonlinear Static Procedure) from ASCE 41-06 is no longer included in ASCE 41-13 as an analysis option because it is often difficult to implement. It is found that NZSEE 2006 Appendix 4E also mentions this simplified NSP.

The use simplified NSP was permitted under the following circumstances:

- Only primary components are modelled, with secondary components excluded from the model. Beside, the acceptance criteria and component demands are also within the scope of only primary components.
- Bilinear component force-deformation relationships are considered, and the degrading portion of the backbone curve is not explicitly modelled.
- The components not meeting the acceptance criteria for primary components are designed as secondary, and are removed from the mathematical model

However, the simplified NSP is no longer applicable, due to the following aspects.

- The removal of the components in the simplified NSP may result in changes to the regularity of the structure that would significantly alter the dynamic response. Otherwise, the secondary components should be included in the model, but may be modelled with negligible stiffness, in order to obtain deformation demands without significantly affecting the overall response.
- The use of the bilinear backbone curves is only permitted when it is proved to be appropriate by post-processing. As the strength degradation is not explicitly modelled in the simplified NSP, the μ_{max} factor cannot be reliably estimated.
- Dynamic instability cannot be properly assessed. Hence, the potential failure mechanisms may be missed, particularly for taller buildings.
- The simplified NSP (in ASCE 41-06)makes it difficult to properly satisfy the requirements of ASCE 41-13, as defining the force-deformation characteristics, primary versus secondary components, and the appropriate acceptance criteria is often challenging and potentially erroneous.
- Only a static load pattern is applied in the simplified NSP. The analysis cannot capture changes in the dynamic characteristics of the structure as yielding and degradation take place.

3.5. Alternative Assessment Procedures

Table 3- 54 gives a brief summary of assessment procedures found in literature. Almost all the codified procedures discussed in the literature are found to be out of dated and have been replaced by the current versions. Only brief descriptions of the alternative procedures, such as DBELA, ATC 40 and ATC 52, etc., are included in the thesis, without showing detailed information.

Table 3- 54: Summary of assessment procedures from literature

Literature	Assessment Procedures
Simplified Approach to Displacement-based Earthquake Loss Estimation Analysis (2002) Development of a Simplified Deformation-Based Method for Seismic Vulnerability Assessment (2003) A Probabilistic Displacement-based Vulnerability Assessment Procedure for Earthquake Loss Estimation, and, Simplified Pushover-based Vulnerability Analysis for Large-Scale Assessment of RC Buildings (2004) Simplified Pushover-based Vulnerability Analysis for Large-scale Assessment of RC Buildings (B. Borzi, R. Pinho, H. Crowley, 2007)	The simplified displacement-based (and probabilistic-based) seismic vulnerability assessment procedure, i.e. DBELA (Displacement-Based Earthquake Loss Assessment Procedure)
Limitations and Performances of Different Approaches for Seismic Assessment of Existing Buildings (G. Lupoi, 2003) Comparison of Different Approaches for Seismic Assessment of Existing Buildings (G. Lupoi and G. M. Calvi, 2004)	• US ASCE (FEMA 356) Prestandard (which is now superseded by ASCE 41-13) • New Zealand Guidelines (Draft Version 2002) (which is now replaced by NZSEE 2006 Guidelines) • Japanese Guidelines (not sure about the latest version)
Analysis of Code Procedures for Seismic Assessment of Existing Buildings (B. Mahaylov, 2006)	• Italian Seismic Code (which is updated by NTC 2008) • FEMA 356, FEMA440 (which is superseded by ASCE 41-13) • EC8 (European Code) (which is updated by EN1998: 2005) • ATC 40 Capacity Spectrum Method applied in HAZUS, basically a NSP
FIB State-of-Art Report: Seismic Assessment and Retrofit of Reinforced Concrete Buildings	• ASCE 1999 (which is now superseded by ASCE 41-13) • New Zealand Guidelines(which is now replaced by NZSEE 2006 Guidelines) • Japanese Guidelines (not sure about the latest version) • EN 1998 (European Code) (which is updated by EN1998: 2005)
ATC 58 Draft Seismic Performance Assessment of Buildings (2011)	Implementation of analysis program PACT Adoption of two alternative structural analysis approaches: • Nonlinear response history analysis • Simplified approach based on elastic analysis and knowledge of structure's yield characteristics

3.5.1. Displacement-Based Earthquake Loss Assessment Procedure (DBELA)

A simplified deformation-based assessment procedure has been developed by Pinho *et al*. since 2002, and has been significantly improved in the past ten years. As specified in this procedure, for a typical building class, from displacement-height relationships and height-period relationships, displacement capacity curves (i.e. displacement-period relationships) under different limit states can be derived. Based on the periods at the intersections of the computed displacement capacity curves and displacement demand spectra, the damage level of the assessed building class can then be determined. The following figure (Figure 3- 23) provides a summary of the procedure. The critical drawbacks of this simplified assessment procedure are shown as following:

- The assessment outcomes are building-class-based, in other words, the response of the individual structure cannot be explicitly assessed.
- The nonlinear response of the structures cannot be appropriately assessed.

Due to the drawbacks of the procedure shown above, it can be argued that this simplified assessment procedure is preferential for the probabilistic-based assessment of a large building inventory.

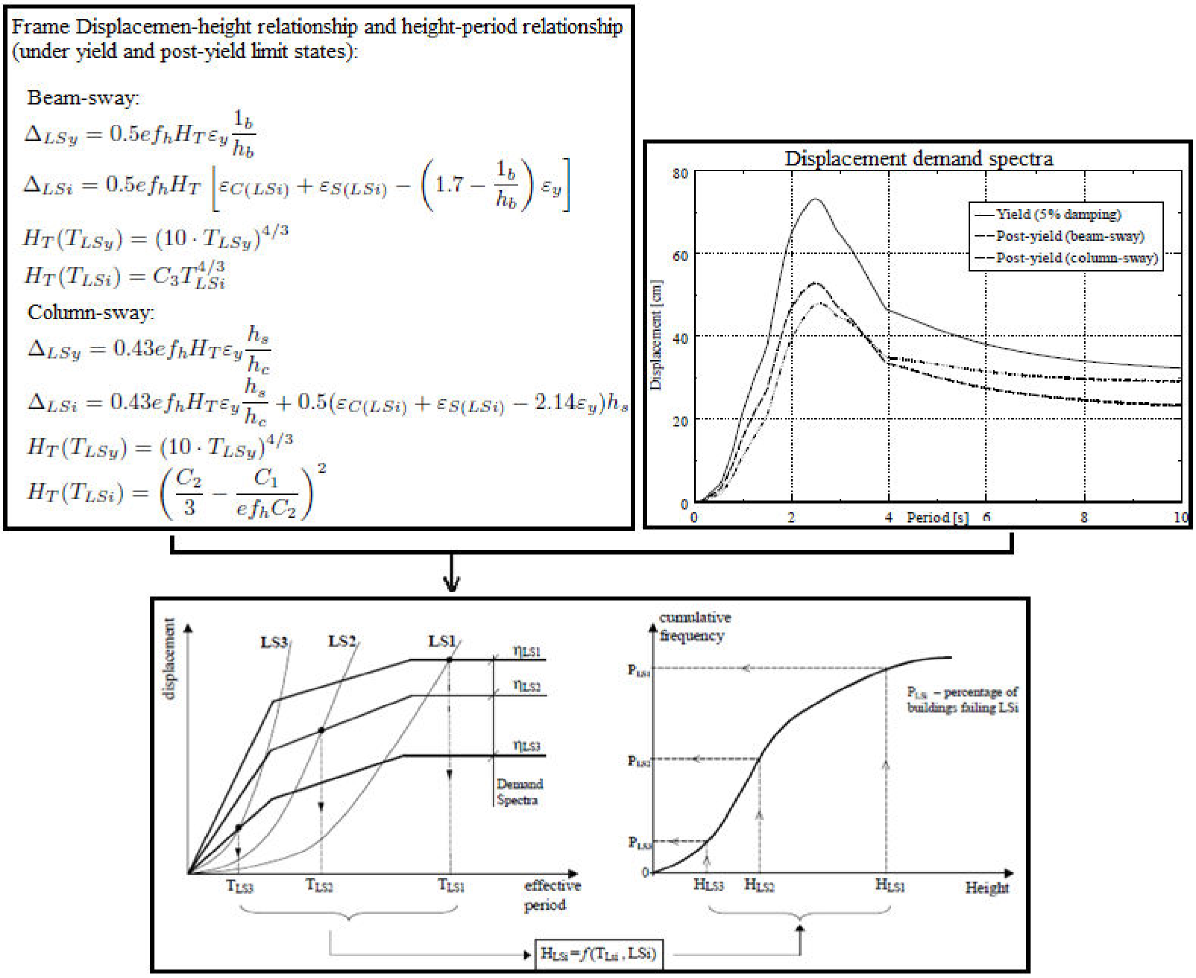


Figure 3- 23: Flowchart of the Simplified Deformation-based Approach Procedure (Pinho et al. 2002, 2003, 2004)

3.5.2. ATC 40 Capacity Spectrum Method

Capacity Spectrum Method, adopted in ATC-40, is one alternative to nonlinear dynamic analysis, and was originally applied in HAZUS methodology for earthquake loss estimation (as stated in B. Borzi, *et al*. 2007). The performance point of the assessed building can be determined from the intersection of an acceleration-displacement spectrum (i.e. demand) and the capacity curve of the building. Different analysis methods, for instances, Nonlinear Pushover Analysis or some simplified analyses, can be applied to determine the capacity curve. It can be concluded that the analysis approaches to determine the capacity is similar to those adopted in the codified procedures, and the most critical difference lies in the determination of the demand. The demand is presented in an acceleration-displacement format (i.e. ADRS), and more discussions regarding applying ADRS are shown in Chapter 9. Figure 3- 24 showing as below, provides an illustration of the determination of performance point.

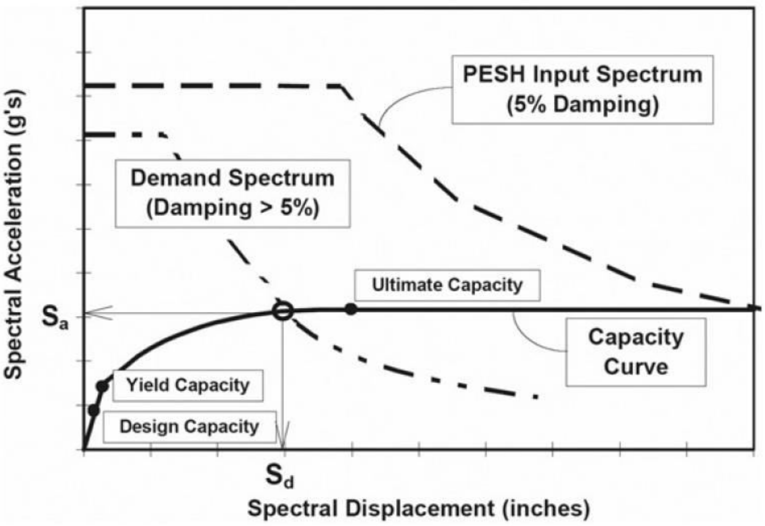


Figure 3- 24: Capacity Spectrum Method, as applied in HAZUS (B. Borzi, et al. 2007)

3.5.3. ATC 58 Application of Analysis Program PACT

In the assessment procedure from ATC 58, two structural analysis approaches are specified: Nonlinear Time History Analysis and a simplified approach based on elastic static analysis. It has been found that the component analysis models applied are similar to those from ASCE 41 but with more specifications regarding cyclic response and degradation of components, as shown in Figure 3- 25, Figure 3- 26 and Figure 3- 27. In modelling, the effects due to geometric nonlinear characteristics, gravity loads, damping, floor diaphragms, soil-foundation-structure interactions, foundation embedment, non-simulated deterioration of components and failure modes are taken into consideration. The analyses can compute good estimates of median values for the key structural response parameters including peak floor acceleration or velocity, peak storey drift and transient drift.

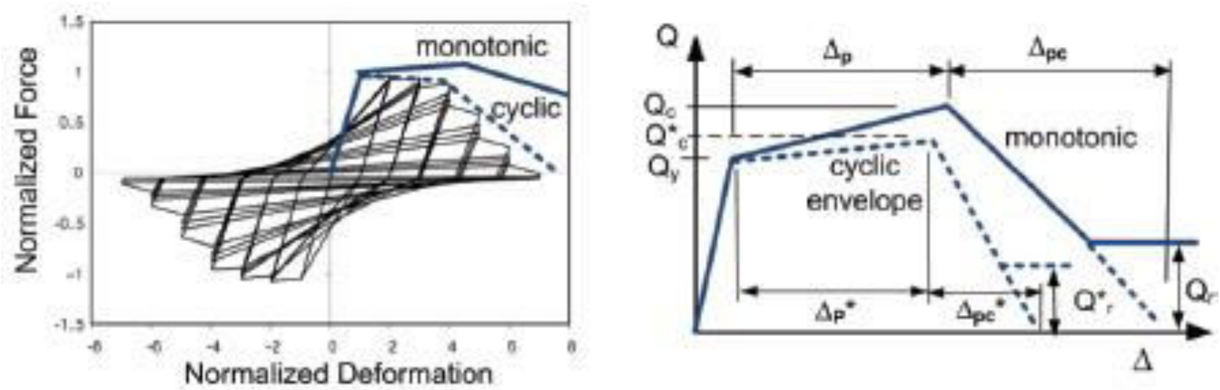


Figure 3- 25: Generalised force-deformation relationships adopted in ATC 58 (ATC 58 Figure 5-1)

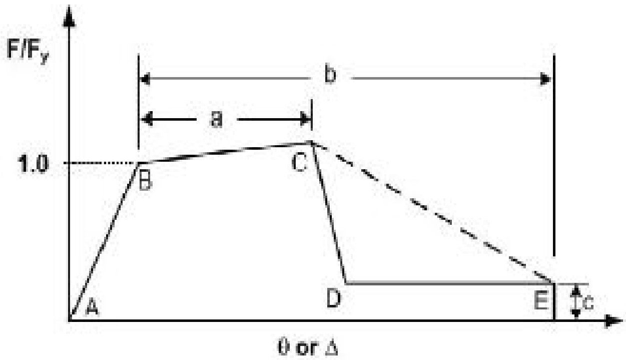


Figure 3- 26: Generalised force-deformation relationship of ASCE 41 (ATC 58 Draft Figure 5-2)

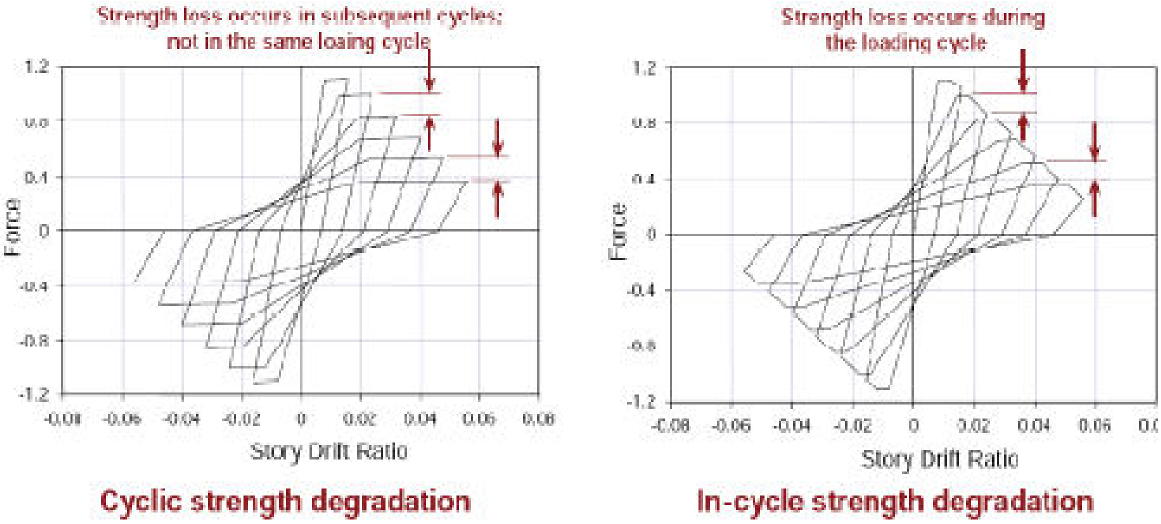


Figure 3- 27: Cyclic versus in-cycle degradation of component response (ATC 58 Draft Figure 5-3)

In ATC-58, a special analysis program, PACT (Performance Assessment Calculation Tool), is applied, and the related information is not included in the thesis.

CHAPTER 4 Overviews and Critical Comparison of Analysis Approaches

4.1. Introduction

This chapter focuses on the analysis approaches applied in the assessment reinforced concrete structures. As shown in Table 3- 52 in Section 3.3.5, the four analysis approaches adopted in all the four codified assessment procedures are Linear Static Analysis (i.e. Equivalent Static Analysis, Lateral Force Analysis, LSP), Linear Dynamic Analysis (i.e. Modal Response Spectrum Analysis, LDP), Nonlinear Static Analysis (i.e. Lateral Pushover Analysis, NSP) and Nonlinear Dynamic Analysis (i.e. Nonlinear Time History Analysis, NDP). From Section 4.2 to 4.5, the differences found for the same type of analysis in different assessment procedures are shown. As mentioned in previous, it has been recognised that NZSEE 2006 includes a unique simplified analytical ("by hand") pushover analysis SLaMa (Simple Lateral Mechanism Analysis), with detailed procedure of this approach is shown in Section 4.6. It is worth noting that in NZSEE 2006 Appendix 4E, the specifications concerning Equivalent Static Analysis, Modal Response Spectrum Analysis, Lateral Pushover Analysis and Nonlinear Time History Analysis are referred to FEMA 356 (which is superseded by ASCE41-13).

4.2. LSP (Linear Static Procedure)

In Table 4- 1, Table 4- 2 and Table 4- 3, the differences lie in the applicability of LSP, the determination of fundamental period, pseudo lateral load, and vertical distribution of seismic forces from the four codified assessment procedures are shown. Table 4- 4 provides a succinct summary of the differences.

From Table 4- 1 in which the applicability or limitations of LSP in the four codified assessment procedures are summarised, the following conclusive comments can be are drawn:

- It is required in all the four codified assessment procedures that LSP is applicable for the buildings with no significant vertical stiffness or mass irregularity. Besides, it is specified in NZSEE 2006 and ASCE 41-13 that the analysis is applicable for the buildings which have insignificant torsional irregularity and orthogonal lateral force resisting system.
- It is required in all the four codified assessment procedures that LSP is applicable for the buildings whose responses are not significantly affected by higher modes. This limitation is associated with constraints of building heights or fundamental periods. It has been found that in the determination of structure height (or effective height) and fundamental period, the

values specified for the parameters or coefficients are different in the four codified assessment procedures, as shown in Table 4- 2.

- It is required in all the four codified assessment procedures that LSP is applicable for the buildings that tend to have elastic responses, in other words, have low ductility. It is worth noting that different criteria are defined in the four procedures. In NZSEE 2006, the ratio of demand to capacity, μ, less than 2 is defined. In ASCE 41-13, the demand-capacity ratio, DCR, less than 3 or m-factor is defined. The determination of DCR is shown in Table 4- 5 and Table 4- 6. In EN 1998-3: 2005 and NTC 2008, ratio ρ is specified by ρi=Di/Ci, where Di is the demand obtained from the analysis under seismic load combination and Ci is the capacity i-th ductile primary element of the structure. EN 1998-3:2005 states that ρmax/ρmin should not exceed a maximum acceptance value in the range of 2 to 3 for ductile mechanism if LSP is applied, while NTC 2008 points out that ρmax/ρmin should be less than 2.5 for ductile mechanisms if LSP is applied.

It is worth noting that most of these limitations of LSP are also applied to LDP (see Section 4.3).

Table 4- 1: Applicability or limitation of LSP defined in the four codified assessment procedures

LSP	NZSEE 2006	ASCE 41-13	EN 1998-3: 2005	NTC 2008
Applicability or Limitations	Building height not exceeding 30 m No significant vertical stiffness or mass irregularity present No significant torsional stiffness irregularity present Orthogonal lateral force resisting systems present *Either:* Elastic responding under design level earthquake *Or:* Low ductility demand/capacity (μ<2) under design level earthquake where: No in plan or out of plan discontinuities present in primary lateral force resisting system No significant weak storey irregularity present No significant torsional strength irregularity present in any storey	Not applicable if a component DCR exceeds the lesser of 3.0 and the m-factor for the component action Not applicable if T ≥ 3.5Ts (Ts = Sx1/Sxs) Not applicable if the ratio of the horizontal dimension at any storey to the corresponding dimension at an adjacent storey exceeds 1.4 (excluding penthouses) Not applicable if the building has a torsional stiffness irregularity in any storey Not applicable if the building has a vertical stiffness irregularity Not applicable if the building has a non-orthogonal seismic-force-resisting system	The fundamental period of vibration T1 doesn't exceed 4 Tc or 2s The buildings must be regular in elevation For the ductile mechanism: the ratio ρmax/ρmin between the maximum and minimum values of ρi(ρi>1) does not exceed a maximum acceptable value in the range of 2 to 3 Applicable to buildings whose response is NOT significantly affected by higher modes contribution in each principal direction.	The fundamental period of vibration T1 doesn't exceed 2.5 TC or TD The buildings must be regular in elevation With elastic spectrum: for ductile mechanism ρmax/ρmin, the maximum and minimum of all ρi≥2, must be less than 2.5; for brittle mechanism ρi must be less than 1

Table 4- 2: Determination of fundamental period in LSP from the four codified assessment procedures

LSP	NZSEE 2006	ASCE 41-13	EN 1998-3:2005	NTC 2008

Period Determination	**Method 1 - Analytical**
	From the dynamic analysis of the mathematical model of the building
	Method 2 - Empirical
	The fundamental period shall be determined in accordance with:
	$T_1 = 1.25k_t h_n^{0.75}$
	kt=0.075 for concrete frames
	=0.11 for steel frames
	=0.06 for eccentrically braced steel frames
	=0.05 for all other frames
	hn=height from the base of the structure to the uppermost seismic weight or mass
	Alternatively for structures with concrete shear walls:
	$k_t = 0.075/\sqrt{A_c}$
	$A_c = \sum[A_i(0.2 + (l_{wi}/h_n)^2)]$
	Ac=total effective area of the shear walls in the first storey
	Ai=effective cross-sectional area of shear wall in 1st storey
	lwi=length of shear wall inthe1st storey in the direction parallel to forces (lwi/hn< 0.9)
	If d is the lateral elastic displacement of the top of the building, in m, due to the gravity loads applied in the horizontal direction:
	$T_1 = 2\sqrt{d}$
	Method 3 – Approximate
	For any building the Rayleigh-Ritz method can be used:
	$T_1 = 2\pi\sqrt{\frac{\sum_{i=1}^{n}(W_i d_i^2)}{g\sum_{i=1}^{n}(F_i d_i)}}$
	di=horizontal displacement at level i, ignoring the effects of torsion
	Fi=displacing force at level i
	Wi=seismic weight at level i
	n=number of levels
	For one-storey buildings with single span flexible diaphragms:
	$T = (3.94U_w + 3.07U_d)^{0.5}$
	U_w=wall displacement(in-plane)
	U_d=diaphragm displacement
	(Due to a lateral load in the direction under consideration)
	For one-storey buildings with multiple-span diaphragms the previous equation may be used as follows: a lateral load equal to the weight tributary to the diaphragm under consideration is applied to calculate a separate period for each diaphragm span. The period that maximizes the pseudo seismic force is used for design of all walls and diaphragm spans in the building
	For unreinforced masonry buildings with single span flexible diaphragms, six storeys or less in height:
	$T = (3.07\,U_d)^{0.5}$
	Ud=maximum in-plane diaphragm displacement due to a lateral load in the direction under consideration
		Method 1 - Analytical
		Eigen value (dynamic) analysis of the mathematical model of the building shall be performed to determine the fundamental period of the building
		Method 2 - Empirical
		The fundamental period of a building, in direction under consideration, shall be calculated:
		$T = C_t h_n^{\beta}$
		C_t=0.0018 for RC frames
		=0.035 for steel frames
		=0.030 for eccentrically braced steel frames
		=0.020 for all other frames
		β=0.90 for RC frames
		=0.80 for steel frames
		=0.75 for all other frames
		h_n=height(ft)above base to roof
		Method 3-Approximate
		Rayleigh's Method or any other rational method to approximate the fundamental period. For the Rayleigh Method:
		$T = 2\pi\sqrt{\frac{\sum_{i=1}^{n}(w_i\delta_i^2)}{g\sum_{i=1}^{n}(F_i\delta_i)}}$
		w_i=portion of effective seismic weight located on or assigned to level i
		δ_i=displacement at floor i caused by lateral force F_i
		F_i=lateral force applied at level i
		n=total number of stories in the vertical seismic framing above base
		For one-story buildings with single-span flexible diaphragms:
		$T = (0.1\Delta_w + 0.078\Delta_d)^{0.5}$
		Δ_w=wall displacement(in-plane)
		Δ_d=diaphragm displacement
		(In inches due to a lateral force in the direction under consideration equal to the weight tributary to the diaphragms)
		For one-story buildings with multi-span flexible diaphragms: a lateral force equal to the weight tributary to the diaphragm span under consideration shall be applied to calculate a separate period for each diaphragm span. The period that maximizes the pseudo seismic force shall be used for analysis of all walls and diaphragm spans in the building
		For unreinforced masonry buildings with single-span flexible diaphragms six stories or fewer high
		$T = (0.078\,\Delta_d)^{0.5}$
		Δ_d=maximum in-plane diaphragm displacement in inches because of a lateral force in the direction under consideration equal to the weight tributary to diaphragm
			Expressions based on methods of structural dynamics (for example the Rayleigh method)
			Or:
			For buildings with heights of up to 40 m :
			$T_1 = C_t H^{3/4}$
			Ct=0.085 steel frames
			=0.075 RC frames& eccentrically braced steel frames
			=0.050 other frames
			H=the height of the building from the foundation or from the top of a rigid basement
			Alternatively for structures with concrete or masonry shear walls:
			$C_t = 0.075/\sqrt{A_c}$
			$A_c = \sum[A_i(0.2 + (l_{wi}//H)^2)]$
			Ac=total effective area of the shear walls in the first storey
			Ai=effective cross-sectional area of shear wall in the direction considered in the 1st storey
			lwi=length of the shear wall in the first storey in the direction parallel to the applied forces (lwi/H < 0.9)
			If d is the lateral elastic displacement of the top of the building, in m, due to the gravity loads applied in the horizontal direction:
			$T_1 = 2\sqrt{d}$
				For constructions that don't exceed 40 m of elevation and that have mass approximately uniformly distributed along the height:
				$T_1 = C_1 H^{3/4}$
				C1=0.085 steel frames
				=0.075 RC frames
				=0.05 other frames
				H =the height of the building

Table 4- 3: Calculation of pseudo lateral load in LSP from the four codified assessment procedures

LSP	NZSEE 2006	ASCE 41-13	EN 1998-3: 2005	NTC 2008
Pseudo Lateral Load	The pseudo lateral load applied to the vertical elements of the lateral force resisting system is: $V = C_1C_2C_3C_mS_aW_t$ C1=modification factor to relate expected maximum inelastic displacements to those calculated for linear elastic response C2=modification factor for pinched hysteresis shape, stiffness, degradation and strength deterioration on the maximum displacement response C3=modification factor for dynamic P-Δ effects Cm=effective mass factor to account for higher mode mass participation factor Sa=response spectrum acceleration at the fundamental period and damping ratio of the building in the direction under consideration Wt=effective seismic weight of the building	The pseudo lateral force in a given horizontal direction of a building shall be determined using equation: $V = C_1C_2C_mS_aW$ C1=modification factor to relate expected maximum inelastic displacements to displacement calculated for linear elastic response C2=modification factor for pinched hysteresis shape, stiffness, degradation and strength deterioration on the maximum displacement response Cm=effective mass factor to account for higher mode mass participation factor Sa=response spectrum acceleration at the fundamental period and damping ratio of the building in the direction under consideration W=effective seismic weight of the building including the total dead load and applicable portions of other gravity loads	The seismic based shear force for each horizontal direction is: $F_b = S_d(T_1)\, m\, \lambda$ Sd(T1)=ordinate of the design spectrum at period T1 m= total mass of the building, above foundation or above top of a rigid basement λ=0.85 if T1 ≤ 2Tc, and the building has more than two storeys =1 otherwise (correction factor)	The seismic based shear force for each horizontal direction is: $F_h = \frac{S_d(T_1)W}{g}\lambda$ Sd(T1)=ordinate of the design spectrum at period T1 W=total weight of building λ=0.85 ifT1<2Tc, and the building has at least three floors =1 otherwise (correction factor)
Vertical Distribution of Seismic Forces	For all buildings except unreinforced masonry buildings, the lateral load at any floor level x is: $F_x = C_{vx}V$ $C_{vx} = \frac{w_x h_x{}^k}{\sum_j w_j h_j{}^k}$ k=2.0 for T ≥ 2.5 s =1.0 for T ≤ 0.5 s And linear interpolation for intermediate values of k wi=portion of the total building weight on floor level i hi=height from base to floor i For unreinforced masonry buildings with flexible diaphragms the pseudo lateral loads can be calculated and distributed as it is written in the Appendix 4E.8.3 $F_{px} = \frac{\sum_{i=x}^{n} F_i}{\sum_{i=1}^{n} w_i} w_x$	For all buildings except unreinforced masonry buildings with flexible diaphragms and seismically isolated structures, the pseudo seismic force shall be distributed vertically as: $F_x = C_{vx}V$ $C_{vx} = \frac{w_x h_x{}^k}{\sum_j w_j h_j{}^k}$ k=1.0 for $T \leq 0.5s$ =2.0 for $T > 2.5s$ And linear interpolation for intermediate value of k wi=portion of the total building weight on floor level i hi = height from base to floor i For unreinforced masonry buildings with flexible diaphragms the pseudo lateral loads can be calculated and distributed as it is written in 7.4.1.3.5	The seismic action effects shall be determined by applying horizontal forces to all storeys: $F_i = F_b \frac{s_i m_i}{\sum s_j m_j}$ where si and sj are the displacements of masses mi and mj in the fundamental mode shape If the fundamental mode shape is approximated by horizontal displacements increasing linearly along the height: $F_i = F_b \frac{z_i m_i}{\sum z_j m_j}$ where zi, zj are the heights of the masses mi mj above the level of application of the seismic action	The horizontal forces over the building height shall be determined by: $F_i = F_h \frac{z_i W_i}{\sum_j z_j W_j}$ where zi and zj are the heights of the masses mi and mj above the level of application of the seismic action

Table 4- 2 provides a summary of the procedures to determine structural fundamental period in LSP from the four codified assessment procedures, and the following conclusive comments can be drawn:

- NZSEE 2006 and ASCE 41-13 provide the most detailed instructions regarding the determination of fundamental period. In both codified assessment procedures, analytical method (e.g. Rayleigh Method), empirical method and approximate Method are specified. The empirical method presented in NZSEE 2006 includes period determination for both frame and shear wall structures, while in ASCE 41-13, only the procedure for frame structures is defined. It has been found that the approximate method in NZSEE 2006 and

ASCE 41-13 are similar, but different values are specified for the parameters in calculation, mainly due to the use of different units (i.e. ft. or in. in ASCE while m in NZSEE).

- EN 1998-3:2005 provides some guidelines regarding analytical method (e.g. Rayleigh Method) and empirical method which are found to be similar to those from NZSEE 2006, though different values are specified for the parameters in calculation.
- NTC 2008 gives the least detailed information regarding period determination, with only empirical method for frame structures presented.

Table 4- 3 provides a summary of the procedures to calculate pseudo lateral load and vertical distribution of seismic forces in LSP from the four codified assessment procedures. It can be concluded that these calculation processes are quite similar, except for the differences in the application of modification factors in calculating pseudo lateral load. It can be recognised that NZSEE 2006 and ASCE 41-13 provide more comprehensive instructions with a number of modification factors applied in order to consider inelastic response, pinched hysteresis shapes, stiffness degradation, strength deterioration, dynamic P-Δ effects, higher mode effect, etc. It has been also found that the calculation of vertical seismic force distribution for masonry buildings is provided by NZSEE 2006 and ASCE 41-13.

The summary table (Table 4- 4) gives concise conclusions of the similarities and differences found in LSP from the four codified assessment procedures.

Table 4- 4: Summary of similarities and differences in LSP from the four codified assessment procedures

<table>
<tr><th>Linear Static Analysis</th><th>NZSEE 2006</th><th>ASCE 41-13</th><th>EN 1998-3: 2005</th><th>NTC 2008</th></tr>
<tr><td>Applicability or Limitations</td><td colspan="4">Similar (building regularity, ratio demand/capacity)</td></tr>
<tr><td>Period Determination</td><td colspan="2">More detailed evaluation (Analytical Method, Empirical Method and Approximate Method)</td><td>Intermediate evaluation (Analytical Method and Empirical Method)</td><td>Basic evaluation (Empirical Method)</td></tr>
<tr><td>Pseudo Lateral Load</td><td colspan="4">Similar but differences in the application of modification factors</td></tr>
<tr><td>Vertical Distribution of Seismic Forces</td><td colspan="4">Similar but slight differences in the coefficients</td></tr>
</table>

In ASCE 41-13, it is found that the LSP applied in Tier 1 is slightly different from that in the higher tiers, with simplified assumptions and calculation process, as shown in Table 4- 5.

Table 4- 5: Comparison between the simplified LSP in Tier 1 and normal LSP in Tier 2 and Tier 3

Simplified LSP in ASCE 41-13 Tier 1	LSP in ASCE 41-13 Tier 2 and Tier 3
$V = CS_aW, \text{alternatively}, V = 0.75W$ $S_a = \frac{S_{X1}}{T} \leq S_{XS}$ $T = C_t h_n^{\beta}, \text{alternatively}, T = 0.10n$ $S_{X1} = F_v S_1, \text{and } S_{XS} = F_a S_S$	$V = C_1 C_2 C_m S_a W$ $C_1 = 1 + \frac{\mu_{strength} - 1}{aT^2}\ (\text{for } T > 1.0s, C_1 = 1.0)$ $C_2 = 1 + \frac{1}{800}\left(\frac{\mu_{strength} - 1}{T}\right)^2\ (\text{for } T > 0.7s, C_2 = 1.0)$ $S_a = \frac{S_{X1}}{T} \leq S_{XS}, (S_{X1} = F_v S_1 \text{ and } S_{XS} = F_a S_S)$ $\mu_{strength} = \frac{S_a}{\frac{V_y}{W}} C_m (\text{Same in Tier 3})$ $\mu_{strength} = \frac{DCR_{max}}{1.5}, C_m \geq 1.0$ $\overline{DCR} = \frac{\sum_1^n DCR_i V_i}{\sum_1^n V_i}, DCR = \frac{Q_{UD}}{Q_{CE}}$
C = Modification factor to relate expected maximum inelastic displacements to displacement calculated for linear elastic response (ASCE 41-13:Table 4-8) S_a = Response spectral acceleration at the fundamental period of the building in the direction under consideration W = Effective seismic weight of the building, including the total dead load and applicable portions of other gravity loads (Details of determining W: ASCE 41-13: 4.5.2.1, 1-4) T = Fundamental period (s) in the direction under consideration C_t = 0.0018 for moment-resisting frames of reinforcement concrete 0.035 for moment-resisting frame systems of steel 0.030 for eccentrically braced steel frames 0.020 for all other framing systems h_n = Height (ft) above the base to the roof level β = 0.90 for moment-resisting frame systems of reinforced concrete 0.80 for moment-resisting frame systems of steel 0.75 for all other framing systems n = Number of stories above the base S_{X1} = Design spectral response acceleration parameter at 1s S_{XS} = Design short-period spectral response acceleration parameter F_a, F_v = Site coefficients (ASCE 41-13:Table 2-4) S_S, S_1 = Response acceleration parameters (ASCE 7: 22)	C_1 = Modification factor to relate expected maximum inelastic displacements to displacement calculated for linear elastic response. C_2 = Modification factor to represent the effect of pinched hysteresis shape, cyclic stiffness degradation, and strength deterioration on maximum displacement response, for buildings with systems that do not exhibit degradation of stiffness and/or strength, C_2=1.0 C_m = Effective mass factor to account for higher modal mass participation effects (ASCE41-13: Table 7-4) (Was developed to reduce the conservatism of the LSP for buildings where higher mode mass participation reduces seismic forces up to 20%) a = Site class factor 130 site Class A or B 90 site Class C 60 site Class D, E, or F (U.S) μ_{streng} = Ratio of elastic strength demand to yield strength coefficient (with elastic base shear capacity substituted for shear yield strength, V_y) V_y = Yield strength of the building in the direction under consideration calculated using results of the NSP for the idealized nonlinear force-displacement curve developed for the building DCR_{ma} = Where the largest DCR (Demand-Capacity Ratio) is computed for any primary component of a building in the direction of response under consideration, taking C_1=C_2=C_m=1.0 $\overline{DCR}$ = Average DCR for elements in the story DCR_i = Critical action DCR for element i of the story V_i = Total calculated lateral shear force in an element i caused by earthquake response, assuming that the structure remains elastic Q_{UD} = Force caused by gravity loads and earthquake forces calculated Q_{CE} = Expected strength of the component or element
No specific procedures to decide horizontal distribution of seismic forces, nor for distribution of seismic forces for unreinforced masonry buildings with flexible diaphragms.	Horizontal distribution of seismic forces: the seismic forces at each floor level of the building shall be distributed according to the distribution of mass at that floor level. Distribution of seismic forces for unreinforced masonry buildings with flexible diaphragms: $F_{px} = \frac{\sum_{i=x}^n F_i}{\sum_{i=1}^n w_i} w_x$ F_{px} = Diaphragm inertial force at level x F_i = Lateral inertial force applied at level i (determined as ASCE 41-13: Equation 7-24, SAME in Tier 1) w_i = Portion of the effective seismic weight W located on or assigned to floor level i w_x = Portion of the effective seismic weight W located on or assigned to floor level x

The characteristics of the simplified LSP in Tier 1 can be summarised as following:

- For short and stiff buildings with low ductility located in the places of High Seismicity, the required building strength may exceed the force required to cause sliding at the foundation level. The strength of the structure, however, does not need to exceed the sliding resistance at the foundation–soil interface. It is assumed that this sliding resistance is equal to 0.75W; thus, the required strength of structural components need not exceed 0.75W.
- For steel or reinforced-concrete moment frames of 12 stories or fewer, the alternative relationship (as shown in Table 4- 2) to approximate the fundamental period is applicable. The values of C_t are intended to be reasonable lower bound but not mean values for structures, including the contribution of nonstructural elements. The value of T should be as close as possible to, but less than, the true period of the structure.
- The analysis performed is only limited to quick checks in Tier 1. Quick checks shall be used to calculate the stiffness and strength of certain building components and to determine whether the building compiles with certain evaluation criteria or not. Table 4- 8 gives a summary of the quick calculation of story drifts of frames, shear stresses in columns and shear walls, and axial stresses caused by overturning in columns. More details associated with investigation on diagonal bracing, precast connections, flexible diaphragm connections, etc. can be found in Chapter 4 and Chapter 16 of ASCE 41-13.

The following aspects of LSP in Tier 2 and 3 are addressed, which are not specified in the simplified LSP.

- As shown in T able 4- 5, DCR shall be calculated for each action, such as axial force, moment, or shear, for each primary component. If a component DCR exceeds the less value of 3.0 and the m-factor for the component action, the linear procedures (LSP and LDP) are not applicable.
- As an alternative to the iterative process of calculating DCR, a simplified way to select appropriate C_1 and C_2 values is suggested. As shown in Table 4- 6, the values should be selected based on fundamental period of the structure and the expected ductility demand based on the maximum m-factor that is permitted for all the primary seismic force resisting system elements. Similarly, Table 4- 7 gives a selection of C_m value.

Table 4- 6: Alternative values for modification factors C_1 and C_2 (ASCE 41-13 Table 7-3)

Fundamental Period	$m_{max} < 2$	$2 \leq m_{max} < 6$	$m_{max} \geq 6$
$T \leq 0.3$	1.1	1.4	1.8
$0.3 < T \leq 1.0$	1.0	1.1	1.2
$T > 1.0$	1.0	1.0	1.1

Table 4- 7: Values for effective mass factor C_m (ASCE 41-13 Table 7-4)

No. of Stories	Concrete Moment Frame	Concrete Shear Wall	Concrete Pier-Spandrel	Steel Moment Frame	Steel Concentrically Braced Frame	Steel Eccentrically Braced Frame	Other
1–2	1.0	1.0	1.0	1.0	1.0	1.0	1.0
3 or more	0.9	0.8	0.8	0.9	0.9	0.9	1.0

NOTE: C_m shall be taken as 1.0 if the fundamental period, T, in the direction of response under consideration is greater than 1.0 s.

- The calculated internal forces typically exceed those that the building can develop due to anticipated inelastic response of components. These forces are evaluated through the acceptance criteria in which modification factors are included, or can be evaluated by applying alternative analysis approaches, to account for the anticipated inelastic response.

Table 4- 8: Examples of quick checks in ASCE 41-13 Tier 1 evaluation

Examples	Calculation procedures
Story drifts for moment-resisting frames (assume that all the columns in the frame have similar stiffness)	$$D_r = \left(\frac{k_b + k_c}{k_b k_c}\right)\left(\frac{h}{12E}\right) V_c$$ D_r = Drift ratio, inter-story displacement divided by story height k_b = I/L for the representative beam k_c = I/L for the representative column h = Story height (in.) I = Moment of inertia (in.^{4}) (effective cracked section moment of inertia equal to ½ of gross value) L = Beam length from centre-to-centre of adjacent columns (in.) E = Modulus of elasticity (kip/in.^{2}) V_c = Shear in the column (kip), calculated using story shear forces (For the 1st floor of the frame, the above equation is applicable if columns are fixed against rotation at the bottom. However, if columns are pinned at the bottom, the drift ratio shall be multiplied by 2.)
Shear stresses in concrete frame columns (assume that an end column carries half of the load of a typical interior column)	$$v_j^{avg} = \frac{1}{M_s}\left(\frac{n_c}{n_c - n_f}\right)\left(\frac{V_j}{A_c}\right)$$ v_j^{avg} = Average shear stress in columns n_c = Total number of columns n_f = Total number of frames in the direction of loading A_c = Summation of the cross-sectional area of all columns in the story under consideration V_j = Story shear computed in previous calculation M_s = System modification factor 2.0 for buildings being evaluated to LS (Life Safety Performance Level) 1.3 for building being evaluated to IO (Immediate Occupancy Performance Level) (This equation is not theoretically correct for a one-bay frame and yields shear forces that are twice the correct force; however, because of the lack of redundancy in one-bay frame, this level of conservatism is considered appropriate.)
Shear stresses in shear walls	$$v_j^{avg} = \frac{1}{M_s}\left(\frac{V_j}{A_w}\right)$$ v_j^{avg} = Average shear stress in shear walls A_w = Summation of horizontal cross-sectional area of all shear walls in the direction of loading (openings shall be taken into consideration where computing A_w) For masonry walls, the net area shall be used. For wood-framed walls, the length shall be used rather than the area. V_j = Story shear computed in previous calculation M_s = System modification factor (ASCE41-13: Table 4-9) 2.0 for buildings being evaluated to LS (Life Safety Performance Level) 1.3 for building being evaluated to IO (Immediate Occupancy Performance Level)
Column axial stresses caused by overturning in columns (assume a triangular force distribution with the resultant applied at 2/3 the height of the building)	$$p_{ot} = \frac{1}{M_s}\left(\frac{2}{3}\right)\left(\frac{Vh_n}{Ln_f}\right)\left(\frac{1}{A_{col}}\right)$$ v_j^{avg} = Axial stress of columns in moment frames at the base subjected to overturning forces n_f = Total number of frames in the direction of loading A_{col} = Area of the end column of the frame V = Pseudo seismic force M_s = System modification factor 2.0 for buildings being evaluated to LS (Life Safety Performance Level) 1.3 for building being evaluated to IO (Immediate Occupancy Performance Level) L = Total length of the frame (ft) h_n = Height (ft) above the base to the roof level

4.3. LDP (Linear Dynamic Procedure)

In Table 4- 9, specifications concerning the aspects of applicability of LDP, number of modes required in analysis, combination rules, damping, from the four codified assessment procedures, are presented.

As shown in Table 4- 9, the applicability of LDP is similar to that of LSP, as mentioned in Section 4.2.

- All the four codified assessment procedures require that the analysis is not applicable for the buildings that tend to have nonlinear responses. The limitations in terms of ductility (i.e. μ from NZSEE 2006), demand/capacity ratio (i.e. DCR from ASCE 41-13), ρ_{max}/ρ_{min} ratio (from EN 1998-3: 2005 and NTC 2008) are similar to those specified in LSP.
- NZSEE 2006 and ASCE 41-13 require that the analysis is applicable for the buildings with no significant irregularity (i.e. in-plane or out-of-plane discontinuity, weak storey irregularity, torsional strength irregularity), while there is no specific limitation defined in EN 1998-3: 2005 and NTC 2008 regarding the issue of irregularity.

The similarities and differences found in the requirements of the number of modes in LDP from the four assessment procedures are summarised as following:

- NZSEE 2006, ASCE 41-13 and EN 1998-3: 2005 require that the number of modes should capture at least 90% of the total building mass for each of directions under consideration, while NTC 2008 requires at least 85% of the total mass. In EN 1998-3: 2005, if the requirement is not satisfied, the minimum number of the modes in a spatial analysis can be calculated by $k \geq 3\sqrt{n}, T_k \leq 0.2$ s, while in the other procedures, no such additional requirement of number of modes for spatial analysis is specified.
- NZSEE 2006 defines the requirements of number of modes for both two-dimensional analysis and three-dimensional analysis, and the requirements are found to be quite similar.

The combination rules applied in LDP from the four assessment procedures are quite similar, and details are shown in Table 4- 9. Generally, a damping ratio of 5% is recommended to use, but more comprehensive guidelines regarding choosing a proper damping ratio in the analysis are provided in ASCE 41-13.

It is worth noting that if force-based linear analyses are used, the evaluation of force reduction factor (q) is a very difficult task. The EN 1998-3:2005gives a conservative assumption that q is equal to 1.5 irrespective of the structural properties, while NTC 2008 suggests that the value of q should be within the range from 1.5 to 3 without explicit choices of exact values.

Table 4- 9: Comparison of LDP from the four codified assessment procedures

LDP	NZSEE 2006	ASCE 41-13	EN 1998-3: 2005	NTC 2008
Applicability or Limitations	Either: Elastic responding under design level earthquake Or: Expected low ductility demand /capacity (μ<2) under design level earthquake where: No in plan or out of plan discontinuities present in primary lateral force resisting system No significant weak storey irregularity present No significant torsional strength irregularity present in any storey	Not applicable if irregularity is defined: in-plane/out-of-plane discontinuity irregularity, weak storey irregularity, torsional strength irregularity. Not applicable if a component DCR exceeds the lesser of 3.0 and the m-factor for the component action.	Buildings which do not satisfy the conditions for applying the lateral force method of analysis For the ductile mechanism: the ratio $\rho max/\rho min$ between the maximum and minimum values of $\rho i(\rho i>1)$ does not exceed a maximum acceptable value in the range of 2 to 3.	If elastic spectrum: for ductile mechanism $\rho max/\rho min$, the maximum and minimum of all $\rho i \geq 2$, must be less than 2.5; for brittle mechanism ρi must be less than 1
Number of Modes	**Two-dimensional analyses** A number of modes that ensure that at least **90%** of total mass of the structure is participating in the direction under consideration. Generally, for vertically and horizontally regular buildings translational modes to be considered should be **half of the number** of storeys but not less than **3** **Three-dimensional analyses** A number of modes that ensure that at least **90%** of the total mass of the structure is participating in each of the two orthogonal directions. In structures modelled so that modes are considered that are not those of the horizontal load resisting systems, all modes not part of the horizontal load resisting systems shall be ignored.	Dynamic analysis using the response spectrum method shall calculate peak modal responses for sufficient modes to capture at least 90% of the participating mass of the building in each of two orthogonal principal horizontal directions of the building	The sum of the effective modal masses for the modes taken into account amounts to at least **90%** of the total mass of the structure and all modes with effective modal masses greater than **5%** of the total mass are taken into account. If the requirements specified cannot be satisfied the **minimum number *k* of modes** to be taken into account in a spatial analysis should satisfy both the two conditions: $k \geq 3\sqrt{n}$ $T_k \leq 0.2$ s k=number of modes considered n= number of storeys above foundation or top of a rigid basement T_k=vibration period of mode k	Each mode with participation mass exceeding the **5 %** and a number of modes with total participation mass exceeding the **85%**
Combination of Modal Action Effects	**Two-dimensional analyses** Either the square root of the sum of the squares (SRSS) or the complete quadratic combination (CQC) technique or any other accepted combination method **Three-dimensional analyses** The complete quadratic combination (CQC) technique or any other accepted combination **Closely spaced modes** If the SRSS is used, the modal action effects from any modes with frequencies within 15% shall first be combined by direct summation ignoring any signs	Peak member forces, displacements, storey forces, storey shears, and base reactions for each mode of response shall be combined by either the Square Root Sum of Squares (SRSS) rule or the Complete Quadratic Combination (CQC) rule	The response in two vibration modes (including both translational and torsional modes) i and j may be taken as independent of each other if: $T_j \leq 0.9T_i$ If all relevant modal responses may be regarded as independent of each other, we can use a Square Root of Sum of Squares combination (SRSS) If it is not satisfied we can use a Complete Quadratic Combination (CQC)	Complete Quadratic Combination(CQC) If the difference between the period of vibration of each mode and the others is at least 10 % the Square Root of Sum of Squares (SRSS) can be used
Damping	Apart from special cases a damping ratio of 5% is to be used	5% damped response spectra shall be used for the analysis of all buildings except those meeting the following criteria: For buildings without exterior cladding, an effective viscous damping ratio, β, equal to 2% of critical damping ($\beta = 0.02$) shall be assumed; For buildings with wood diaphragms and cross walls that interconnect the diaphragm levels at a maximum spacing of 40 ft on centre transverse to the direction of motion, an effective viscous damping ratio, β, equal to 10% of critical damping ($\beta = 0.10$) shall be permitted; For buildings using seismic isolation technology or enhanced energy dissipation technology, an equivalent effective viscous damping ratio, β, shall be calculated using the procedures specified in Chapter 14; or There is sufficient analysis or test data based on the specific characteristics of the building to substantiate the use of a damping ratio other than 5% ($\beta = 0.05$)	Apart from special cases a damping ratio of 5% is to be used	Apart from particular cases as constructions with base isolation system or dissipative system, we can assume that the modes of vibration have the same value of conventional damping which is 5%

Table 4- 10 gives a brief summary of the similarities and differences found in LDP from the four procedures.

Table 4- 10: Summary of similarities and differences in LDP from the four codified assessment procedures

Linear Dynamic Procedure	NZSEE 2006	ASCE 41-13	EN 1998-3: 2005	NTC 2008
Applicability or Limitations	Similar (demand/capacity ratio, irregularity)			
Number of Modes	Basic (total participating mass)		More details in calculation of number of modes in spatial analysis	Basic (total participating mass + effective modal mass)
Combination of Modal Action Effects	Similar (SRSS, CQC)			
Damping	Basic	More details in determination of damping ratio	Basic	Basic

4.4. NSP (Nonlinear Static Procedure)

In Table 4- 11 to Table 4- 16, specifications concerning the aspects of applicability of NSP, determination of control node, capacity curve, period, damping ratio, lateral load, and target displacement, from the four codified assessment procedures, are presented.

Table 4- 11: Applicability or limitation of NSPfrom the four codified assessment procedures

NSP	NZSEE 2006	ASCE 41-13	EN 1998-3: 2005	NTC 2008
Applicability or Limitations	Higher mode effects not critical (it is difficult to incorporate higher mode effect so in most cases it is still essentially a single mode approach and collapse mechanisms associated with higher modes may be missed) A difficulty with the lateral pushover analysis is that a static representation of the distribution of the seismic forces acting on the frame is required. (Details in Determination of Lateral Load table) Most LPA (Lateral Pushover Analysis) programs cannot deal with negative structural stiffness, so it can be difficult to determine structural displacement capacity.	Not applicable when higher mode effects are significant. However, if higher modes effects are significant NSP shall be acceptable in conjunction with the LDP if the mass participation in the first mode is low and the building must meet the acceptance criteria for both analysis procedures, except that an increase by a factor of 1.33 shall be permitted in the LDP acceptance criteria for deformation-controlled action (m-factors) Not applicable if the strength ratio $\mu_{strength}$ calculated exceeds μ_{max}	Pushover analysis performed with the force patterns may significantly underestimate deformations at the stiff/strong side of a torsional flexible structure. The same applies for the stiff/strong side deformations in one direction of a structure with a predominately torsional second mode of vibration. For such structures, displacements at the stiff/strong side shall increase compared to those in the corresponding torsional balanced structure	Buildings with a behaviour under earthquake governed by a principal mode of vibration with a significant participation mass This method can be used if conditions of distributions belonged to Group 1 are satisfied. The analysis may underestimate significantly deformations on the sides more rigid and resistant of torsional flexible structures

As shown in Table 4- 11, applicability or limitations of NSP specified in the four codified assessment procedures are found to be similar, and conclusive comments are made as follows.

- It is required in all the four assessment procedures that NSP is applicable when higher mode effects are insignificant. In addition, ASCE 41-13 suggests that if the higher mode effects cannot be ignored, the analysis can be carried out in conjunction with LDP, with some additional requirements show in Table 4- 11.

- NZSEE 2006, EN 1998-3:2005 and NTC 2008 address difficulties and potential issues associated with the application of static force distribution in the analysis. The application of inappropriate load pattern may lead to an underestimation of deformation. More details regarding this issue are discussed in the following paragraphs.
- It is stated in NZSEE 2006 that negative structural stiffness (i.e. degradation of strength) may not be properly considered and modelled in the analysis; hence, this issue may leads to erroneous estimation of lateral force and displacement capacities.
- As discussed in Section 3.3.4, ASCE 41-13 adopts component force-deformation curves (i.e. component models) together with comprehensive specifications of modelling parameters and numerical acceptance criteria. Computer analysis tools with the component models implanted may be used if available.

It can be found that the definitions of control node displacement specified in the four codified assessment procedures are similar – at the centre of the mass at the building roof (or floor of the penthouse if it exists), as shown in Table 4- 12.

Table 4- 12: Control node displacement defined in NSP from the four codified assessment procedures

NSP	NZSEE 2006	ASCE 41-13	EN 1998-3: 2005	NTC 2008
Control Node Displacement	The control node should be located at the centre of the mass at the roof of a building For buildings with a penthouse, the floor of the penthouse should be regarded as the level of the control node	The control node shall be located at the centre of mass at the roof of a building. For buildings with a penthouse, the floor of the penthouse should be regarded as the level of the control node	The control node should be located at the centre of the mass at the roof of a building The top of a penthouse should not be considered as the roof	The control node should be located at the centre of the mass of the last level of a building

In Table 4- 13, the critical procedures to determine capacity curve in NSP from the four codified assessment procedures are presented, and the differences found are addressed as following:

- Apart from NTC 2008, in the other three assessment procedures, it is required that the capacity curve should be established for the control node displacement ranging from 0 to 150% of the target displacement.
- EN 1998-3: 2005 and NTC 2008 assume an idealised elasto-perfectly plastic force-displacement relationship as capacity curve of an equivalent SDOF. NZSEE 2006 suggests a bilinear relationship with initial stiffness Ke and post-yield stiffness α. ASCE 41-13, however, suggests a three-segment-relationship with initial stiffness Ke, positive post-yield stiffness α_1Ke and negative post-yield stiffness α_2Ke.

Table 4- 13: Determination of capacity curve in NSP from the four codified assessment procedures

NSP	NZSEE 2006	ASCE 41-13	EN 1998-3: 2005	NTC 2008
Capacity Curve	The capacity curve of the control node should be established for control node displacements ranging between **zero and 150%** of the target displacement. The nonlinear force-displacement relationship between the base shear and the displacement of the control node shall be replaced with an **idealised relationship** to calculate the effective lateral stiffness, Ke, and effective yield strength, Vy , of the building This relationship shall be **bilinear**, with initial slope **Ke** and post-yield slope **α**. Line segments on the idealised force-displacement curve shall be located using an iterative graphical procedure that approximately balance the **area** above and below the curve. The effective lateral stiffness, Ke , shall be taken as the secant stiffness calculated at a base shear force equal to **60%** of the effective yield strength of the structure. The post-yield slope, α, shall be determined by a line segment that passes through the actual curve at the calculated target displacement. The effective yield strength shall not be taken as greater than the maximum base shear force at any point along the actual curve.	The relation between base shear force and lateral displacement of the control node shall be established for control node displacements ranging between **0 and 150%** of the target displacement δt. The nonlinear force-displacement relationship between base shear and displacement of the control node shall be replaced with an **idealised relationship** to calculate the effective lateral stiffness, Ke, and effective yield strength, Vy The first Line segment of the idealized force-displacement curve goes from (0, 0) with a slope **Ke**, where Ke shall be taken as the secant stiffness calculated at a base shear force equal to**0.6Vy**. The effective yield strength Vy shall not be taken as greater than the maximum base shear force at any point along the force-displacement curve The Line segment 2: with positive post-yield slope **α_1Ke**, determined by (**Vd**, Δd) and the intersection point with line segment 1, so that the **areas** above and below the actual curve are approximately balanced The Line segment 3: with negative post-yield slope **α_2Ke**, determined by (Vd, Δd) and the point at which the base shear degrades to **0.6Vy**	The capacity curve should be determined by pushover analysis for values of the control displacement ranging between **zero** and the value corresponding to **150%** of the target displacement We can determine the **idealized elasto-perfectly plastic force-displacement relationship** as follows: the yield force Fy*, which represents also the ultimate strength of the idealized system, is equal to the base shear force at the formation of the plastic mechanism. The **initial stiffness** of the idealized system is determined in such a way that the **areas** under the actual and the idealized force deformation curves are equal. Based on this assumption we can define the yield displacement of the idealized SDOF system dy*	An equivalent structural system of SDOF must be associated with the structure. The capacity curve of the equivalent system may be replaced by a **bilinear** curve with a first **elastic** line segment and a second **perfectly plastic** line segment. If Fbu* is the maximum resistance of the equivalent system, the first line is obtained requiring the passage for the point **0.6Fbu*** of the capacity curve of the equivalent system. The yield force Fy* is identified by imposing the equality of the areas under the bilinear curve and the capacity curve for the maximum displacement du* corresponding to a reduction of resistance ≤ 0,15Fbu*

Table 4- 14: Determination of period and damping ratio in NSP from the four codified assessment procedures

NSP	NZSEE 2006	ASCE 41-13	EN 1998-3: 2005	NTC 2008
Period Determination	The effective fundamental period in the direction under consideration shall be based on the idealised force-displacement curve defined: $T_e = T_i\sqrt{\frac{K_i}{K_e}}$ T_i = Elastic fundamental period (in seconds) in the direction under consideration calculated by elastic dynamic analysis K_i = Elastic lateral stiffness of the building in the direction under consideration K_e = Effective lateral stiffness of the building in the direction under consideration	Based on the idealised force-displacement curve, the effective fundamental period: $T_e = T_i\sqrt{\frac{K_i}{K_e}}$ T_i = Elastic fundamental period (in seconds) in the direction under consideration calculated by elastic dynamic analysis K_i = Elastic lateral stiffness of the building in the direction under consideration (ASCE 41-13 Chapter 8 – 12 and 14) K_e = Effective lateral stiffness of the building in the direction under consideration	The period T*of the idealised equivalent SDOF system is determined by: $T^* = 2\pi\sqrt{\frac{m^* d_y^*}{F_y^*}}$	The elastic period T*of the bilinear system is determined by: $T^* = 2\pi\sqrt{\frac{m^*}{k^*}}$ where k* is the stiffness of the elastic segment
Damping	Same as those in linear analyses	Same as those in linear analyses	Same as those in linear analyses	Same as those in linear analyses

In Table 4- 14, specifications regarding the determination of period and damping ratio in NSP from the four codified assessment procedures are shown. In NZSEE 2006 and ASCE 41-13, effective period is determined based on idealised force-displacement curve, and is calculated from elastic fundamental period and elastic lateral stiffness. While in EN 1998-3: 2005 and NTC 2008, the period of an equivalent system is determined from its mass and elastic stiffness. It is also worth noting that the requirements of damping ratio applied in NSP are same with those specified in the linear analyses (see Section 4.2 and 4.3).

In Table 4- 15, specifications regarding the determination of lateral load in NSP from the four codified assessment procedures are shown. It is recommended in NZSEE 2006, EN 1998-3: 2005 and NTC 2008 that at least two distributions of lateral forces should be applied in NSP, at least one from "modal"–pattern group and at least one from "uniform"–pattern group. However, ASCE 41-13 suggests that only a single pattern based on the first mode shape should be applied, explaining that the application of multiple force patterns may not make any difference to improve the accuracy of the analysis results. This leads to a major confliction between ASCE 41-13 and the others, and definitely requires future investigation. The two groups of load patterns specified in NZSEE 2006 are shown following Table 4- 15.

Table 4- 15: Determination of lateral load in NSP from the four codified assessment procedures

NSP	NZSEE 2006	ASCE 41-13	EN 1998-3: 2005	NTC 2008
Lateral Load	The lateral load should be applied in both the positive and negative directions and the maximum seismic effects should be used for design. Allows choice of the shape of lateral force vector (most choose inverted triangle but a structure with soft-story sway mechanism should have a force vector essentially uniform with height) (Conventionally an inverted triangular distribution of lateral seismic forces up to the height of the frame could be assumed, but this distribution takes no account of higher mode effects. A sensitive analysis may need to be conducted assessing the differences in lateral force capacity V of the frame arising from different distributions of seismic load, e.g. uniform up the height (which will be of particular interest for taller structures when higher modes will become important) At least two vertical distribution of lateral load should be applied. One pattern shall be selected from each of the two groups described in the Appendix 4E.11.3 Lateral load distribution from modal analysis (LDP) can be applied to provide some allowance for higher modes but will only be completely valid while the structure remains predominantly in the elastic range.	The seismic forces shall be applied in both the positive and negative directions and the maximum seismic effects should be used for the analysis. Lateral loads shall be applied to the mathematical model in proportion to the distribution of mass in the plane of each floor diaphragm. The vertical distribution of these forces shall be proportional to the shape of the fundamental mode in the direction under consideration. (The actual distribution is expected to vary continuously during earthquake response as portions of the structure yield and stiffness change. Research in FEMA 440 suggests that multiple force patterns do little to improve the accuracy of nonlinear static procedures and that a single pattern based on the first mode shape is recommended.)	The seismic action shall be applied in both positive and negative directions and the maximum seismic effects as a result of this shall be used. At least two vertical distributions of the lateral loads should be applied: - a "uniform" pattern, based on lateral forces that are proportional to mass regardless of elevation (uniform response acceleration) - a "modal" pattern, proportional to lateral forces consistent with the lateral force distribution in the direction under consideration determined in elastic analysis The plastic mechanism shall be determined for the two lateral load distributions applied. The plastic mechanisms shall conform to the mechanisms on which the behaviour factor q used in the design is based.	The lateral load should be applied in both the positive and negative directions and the maximum seismic effects should be used for design. At least two distributions of inertia forces must be considered, one pattern selected from the principal direction (Group 1) and one pattern selected from the secondary distribution (Group 2) (7.3.4.1)

A modal pattern selected from one of the following:

- A vertical distribution proportional to the values of C_{vx} (i.e. vertical distribution of seismic forces)– this distribution should be used only when more than 75% of the total mass participates in the fundamental mode in the direction under consideration, and should ne used together with the uniform distribution.
- A vertical distribution proportional to the shape of the fundamental mode in the direction under consideration–this distribution should be used only when more than 75% of the total mass participates in this mode.
- A vertical distribution proportional to story shear distribution– this distribution should be used only when the period of the fundamental mode exceeds 1.0s. The story shear distribution should be calculated by combining modal responses from a response spectrum analysis of the building, including sufficient modes to capture at least 90% of the total building mass, and using the appropriate ground motion spectrum.

A second pattern selected from one of the following:

- A uniform distribution with lateral forces at each level proportional to the total mass at each level
- An adaptive load distribution that varies as the structure is displaced.
  Procedures for developing adaptive load patterns include the use of story forces proportional to the deflected shape of the structure (Fajfar and Fischinger), the use of load patterns based on mode shapes derived from secant stiffness at each load step (Everhard and Sozen), and the use of load patterns proportional to the story shear resistance at each step (Bracci*et al.*). Use of an adaptive load pattern will require more analysis effort, but may yield results that are more consistent with the characteristics of the building under consideration.

As stated in NZSEE 2006, the distribution of lateral inertial forces determines relative magnitudes of shears, moments, and deformations within the structure, and will vary continuously during earthquake response as portions of the structure yield and stiffness characteristics change. The extremes of this distribution will depend on the severity of the earthquake shaking and the degree of nonlinear response of the structure.

One of the main reasons to use more than one lateral load pattern in NSP is that the application of multiple load distributions can help to bound the range of design actions that may occur during actual dynamic response.

Table 4- 16: Determination of target displacement in NSP from the four codified assessment procedures

NSP	NZSEE 2006	ASCE 41-13	EN 1998-3 : 2005	NTC 2008
Target Displacement	The target displacement is at each floor level can be calculated in accordance with: $\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{4\pi^2} g$ C_0 = Modification factor to relate spectral displacement of an equivalent single-degree-of-freedom (SDOF) system to the roof displacement of the building multi-degree-of-freedom (MDOF) system calculated using one of the following procedures: The first modal participation factor at the level of the control node; the modal participation factor at the level of the control node calculated using a shape vector corresponding to the deflected shape of the building at the target displacement. This procedure shall be used if the adaptive load pattern is used; or the appropriate value from Table 3-2 of FEMA 356 - 2000 (Other parameters seen in LSP)	The target displacement shall be no less than: $\delta_t = C_0 C_1 C_2 S_a \frac{T_e^2}{4\pi^2} g$ (for rigid diaphragms shall be amplified by the ratio of the maximum displacement at any point on the roof to the displacement at the centre of mass of the roof) C_0 = Modification factor to relate spectral displacement of an equivalent single-degree-of-freedom (SDOF) system to the roof displacement of the building multi-degree-of-freedom (MDOF) system calculated using one of the following procedures: The first mode mass participation factor multiplied by the ordinate of the first mode shape at the control node; The mass participation factor calculated using a shape vector corresponding to the deflected shape of the building at the target displacement multiplied by ordinate of the shape vector at the control node; The appropriate value from ASCE 41-13 Table 7-5 (Other parameters seen in LSP) The target displacement shall be modified to consider the effects of torsion, i.e. multiplied by factor η	The target displacement is the seismic demand derived from the elastic response spectrum in terms of the displacement of an equivalent SDOF system The target displacement of the structure with period T* and unlimited elastic behaviour is given by: $d_{et}^* = S_e(T^*)(T^*/(2\pi))^2$ Where$S_e(T^*)$is the elastic acceleration response spectrum at the period T*. For the determination of the target displacement d_t^* for structures in the short-period range and for structures in the medium and long-period ranges different expressions should be used (see Appendix B5)	To find the displacement demand the structural system is studied as an equivalent SDOF If the elastic period of the construction is T*≥Tc the displacement demand is assumed equivalent to that of an elastic system with the same period: ($d_{max}^* = d_{e,max}^* = S_{De}(T^*)$) If T*< Tc the displacement demand $d_{max}^*$is given by: $\frac{d_{e,max}^*}{q^*}\left[1 + \frac{(q^* - 1)T_c}{T^*}\right]$ q*=ratio between the elastic force and the yield force of the equivalent system

In Table 4- 16, specifications regarding the determination of target displacement in NSP from the four codified assessment procedures are shown. NZSEE 2006 and ASCE 41-13 adopt similar procedures to determine target displacement in the analysis, except for the differences in values specified for some parameters or coefficients. However, EN 1998-3: 2005 and NTC 2008 suggest different ways to calculate target displacement, as shown in Table 4- 16.

The following table (Table 4- 17) summarises the similarities and differences of NSP from the four codified assessment procedures. As mentioned in Section 3.4, ASCE 41-06 includes a simplified nonlinear static analysis, but is abandoned in ASCE 41-13. Discussions regarding this non-applicable simplified NSP are shown in Section 3.4. NZSEE 2006 Appendix 4E also includes this simplified nonlinear static analysis.

Table 4- 17: Summary of similarities and differences in NSP from the four codified assessment procedures

Nonlinear Static Procedure	NZSEE 2006	ASCE 41-13	EN 1998-3:2005	NTC 2008
Applicability or Limitations	Similar but ASCE introduces a limitation for the strength ratio			
Control Node Displacement	Similar (centre of mass at the roof of the building)			
Capacity Curve	Bi-linear relationship with pre-yield slope and post-yield slope	Three line segments	Bi-linear elasto-plastic relationship	
Period Determination	Effective period		Elastic period	
Lateral Load	Combination of Modal pattern and Second pattern	Modal pattern	Modal and uniform pattern	Combination of Group 1 and Group 2
Target Displacement	More coefficients		Basic (SDOF)	
Damping	Same in the linear procedures			

4.5. NDP (Nonlinear Dynamic Procedure)

In Table 4- 18, the applicability or limitations of NSP specified the four codified assessment procedures are shown. Theoretically, NDP can be used for any structures, regardless of constraints due to analysis tools and artificial factors. NZSEE 2006 and ACE 41-13 requires that special care and skills (e.g. review and approval by an independent third-party engineer with experience) are necessary to carry out the analysis. In addition, NZSEE 2006 also addresses some limitations regarding modelling, for instance, the modelling of interactions of flexure, shear, axial loads, the modelling of strength degradation, and the modelling of vulnerable beam-column-joint, etc. In Table 4- 19, details of modelling characteristics from the four assessment procedures are provided. The summary table (Table 4- 20) of the similarities and differences in the four assessment procedures is presented in the end of this section.

Table 4- 18: Applicability or limitation of NDP specified in the four codified assessment procedures

NDP	NZSEE 2006	ASCE 41-13	EN 1998-3: 2005	NTC 2008
Applicability or Limitations	May be used for any structure but may be not appropriate for some structures, e.g. wooden framed structures. Special care and skill is required to select appropriate modelling approximation. For example, the definition of elastic damping needs careful consideration, as inappropriate definition commonly results in an overestimate of response. Typically the interactions of flexure, shear and axial load are typically not modelled in ITHA programs, making it impossible to model the onset of shear failure. Similarly, few ITHAs include the influence of axial force in columns on their stiffness. This can influence predictions of onset of inelastic response, and can be critical for structures with brittle failure modes. Some ITHA programs cannot model degrading strength characteristics, and few have special elements representing the strength and degradation characteristics of beam-column joints in concrete or steel structures.	The NDP shall be permitted for all structures. When the NDP procedure is used the authority having jurisdiction shall consider the requirement of review and approval by an independent third-party engineer with experience in seismic design and nonlinear procedures.		

Table 4- 19: Modelling characteristics specified in NDP from the four codified assessment procedures

NDP	NZSEE 2006	ASCE 41-13	EN 1998-3 : 2005	NTC 2008
Control	It should be supported by the results of a simplified approach.			It must be compared with a modal analysis with design spectrum to control the differences in terms of global stresses at the base of the structure.
Building Response	The most critical value of any response parameter (e.g. stress, strain, rotation, displacement) across the family of records shall be used to determine acceptability. The design horizontal deflections shall be taken as the maxima of the appropriate deflections obtained for each of the required ground motions. The design inter-storey deflection between adjacent levels shall be taken as the maximum of the inter-storey deflections obtained for each of the required ground motions.	<u>Component response is independent of the direction of action:</u> Average component actions (incl. forces and deformations) shall be calculated as the mathematical mean of the maximum absolute response from each response history analysis. The maximum response shall be calculated as the maximum absolute response from each response history analysis. <u>Component response is dependent on the direction of action:</u> average response parameter shall be calculated independently for each direction and axis as the mathematical means of the maximum positive and minimum negative response from each response history analysis. The maximum response parameter shall be determined independently for each direction of action as the maximum positive and minimum negative response from each response history analysis. To consider torsional effect, the amplitude of the ground acceleration record shall be amplified by the maximum value of factor η	If the response is obtained from at least 7 nonlinear time-history analyses with ground motions the average of the response quantities from all of these analyses should be used as the design value of the action effect in the verifications. Otherwise, the most unfavourable value of the response quantity among the analyses should be used.	If we use 7 different families of records the average of the response quantities from all of these analyses should be used as the design value of the action effect in the verifications. Otherwise, the most unfavourable value of the response quantity among the analyses should be used
Damping	Viscous damping of 5% for all modes whose period is less than the analysis time step included in the analysis is to be used unless a different value is recommended by the appropriate material Standard If Rayleigh damping is used, there shall be no more than 5%of critical damping in the two first translational modes, and no more than 40% damping in the mode with the period T_n	Damping can be modelled using Rayleigh damping or other rational methodology. Target elastic viscous damping ratio shall not exceed 3% (β=0.03), except when: Building without exterior cladding, the target effective elastic viscous damping ratio shall not exceed 1% (β=0.01) Higher values are permitted if substantiated through analysis or test data Special care require where damping is implemented using mass and stiffness proportional methods (The lower damping limits associated with the NDP relative to the linear and nonlinear static procedures account for the explicit modelling of hysteretic damping in the analysis. The damping ratio should be limited to no greater than the target equivalent viscous damping ratio at long period)		

Table 4- 20: Summary of similarities and differences in NDP from the four codified assessment procedures

Nonlinear Dynamic Procedure	NZSEE 2006	ASCE 41-13	EN 1998-3 : 2005	NTC 2008
Applicability or Limitations	Similar(for all structures)			
Control	Similar(supported by the results of simplified approaches)			
Building Response	Maximum response	Average response(response independent or dependent on the direction of action)	Average response or Maximum response	Average response or Maximum response
Damping	Basic	More detailed	None	

4.6. SLaMa (Simplified Lateral Mechanism Analysis)

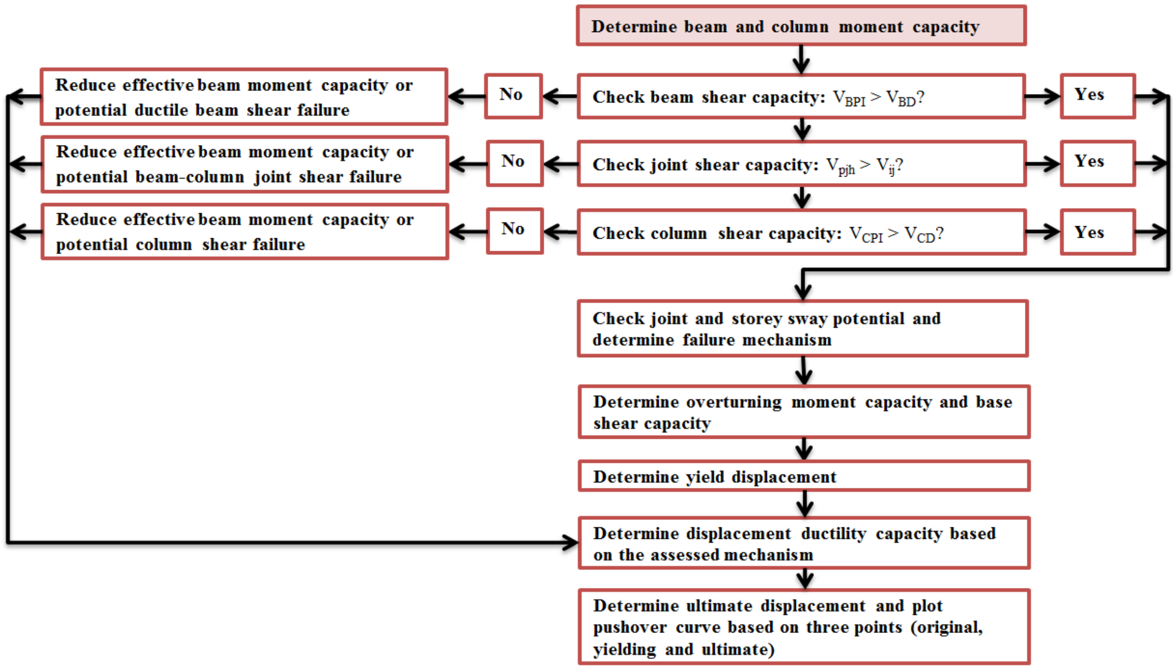
4.6.1. Introduction

SLaMa is referred as an analytical ("by-hand") nonlinear approach to compute pushover curve, and is unique in NZSEE 2006. The flowchart of procedure is shown in Figure 4- 1. It is worth noting that NZSEE 2006 only includes the step-by-step SLaMa procedure for reinforced concrete frame-type structures, and it has been argued that the procedure needs to be improved according to the most advanced research. Hence, investigations and researches are required to propose improvements and also to develop simplified procedures to assess other types of structures. The proposed improvements are discussed in Chapter 5, and in Section 5.7, a very brief introduction of SLaMa applied to shear wall structures is presented.

Figure 4- 1: Flowchart of SLaMa procedure (RC frames) in NZSEE 2006

4.6.2. Procedures (for RC Frames)

4.6.2.1. Determine Beam and Column Moment Capacity



Yielding and ultimate moments of beam and column sections of all levels can be computed by hand calculation using strain capability assumption and stress block theory illustrated in Figure 4- 2, or can be approximated by the preliminary design method defined in SNZ 1995 (i.e. NZS3101:1995) (i.e. $A_sf_yj_d$). The curvatures at three critical points – cracking point, yielding point and ultimate point – should be calculated, and a bilinear moment-curvature relationship can then be computed. It should be recognised that the flexural capacity of a section should have a varying range with the nominal strength as the lower bound and the overstrength as the upper bound. More discussions regarding the variation of flexural capacities are presented in Chapter 7 and Chapter 9.

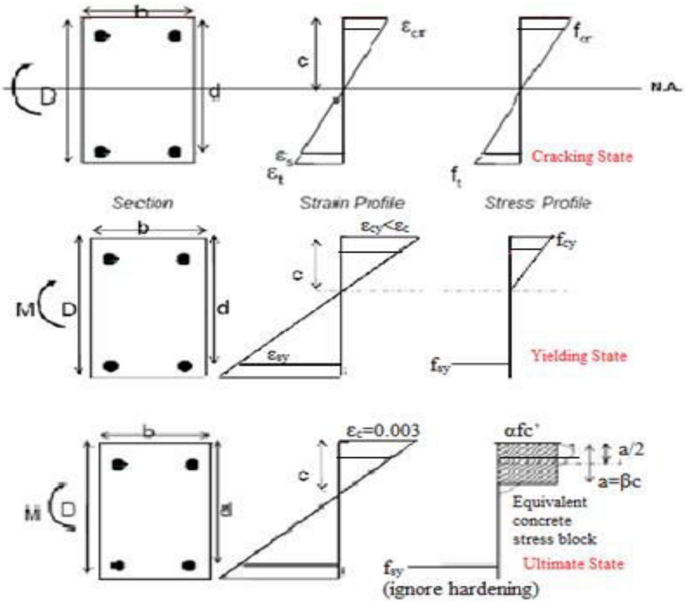
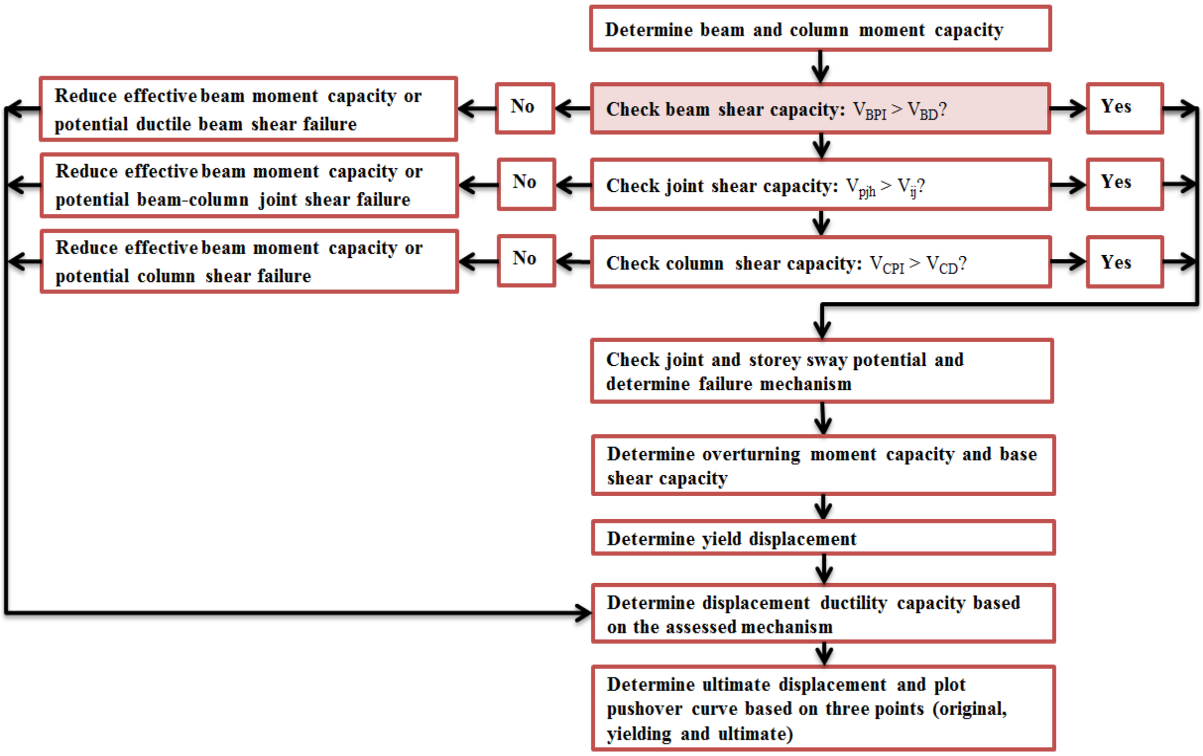


Figure 4- 2: Strain-stress Relationships at Cracking, Yielding and Ultimate States

The following problems can be addressed when carrying out SLaMa at this stage (i.e. determining beam and column moment capacity), and the measures to solve these problems are discussed in Chapter 5

- There are no detailed instructions of computing moment-curvature relationships.
- There are no instructions to estimate flexural strength of irregular sections by hand calculation.
- There are no specific guidelines regarding the impact of axial load variation on columns.

4.6.2.2. Check Beam Shear Capacity



Check if $V_{BPI} > V_{BD}$ (capacity > demand):

$$V_{BPI} = 0.85\left(v_c b_w d + \frac{A_v f_{yt} d}{s}\right) = 0.85\left(k\sqrt{f_c'} b_w d + \frac{A_v f_{yt} d}{s}\right)$$

V_{BPI}	=	Initial probable beam shear strength
v_c	=	Nominal shear stress carried by concrete mechanism
k	=	0.2 (reduced value considering cyclic loading)
f_c'	=	Expected concrete compressive strength
b_w	=	Width of beam web
d	=	Effective depth of beam
A_v	=	Area of transverse shear reinforcement at spacing s
f_{yt}	=	Expected yield strength of the shear reinforcement
s	=	Spacing of transverse shear reinforcement

$$V_{BDl} = V_{BGl} + \frac{M_{BNl} + M_{BNr}}{L_{bc}}$$

V_{BD}	=	Beam shears at the moment capacities (V_{BDl}: beam shear at left side of beam)
V_{BG}	=	Beam gravity shear forces (V_{BGl}: gravity shear at left side of beam)
M_{BN}	=	Beam moment capacity (M_{BNl}: moment capacity at left side of beam and M_{BNr}at right side)
L_{bc}	=	Beam clear span

Beam shear capacity can be estimated considering both concrete contribution and structural steel contribution, referring to SNZ 1995 (i.e. NZS3101:1995). It is worth noting that the value of coefficient k needs to be reduced considering cyclic loading, according to Figure 4- 3. The value of k should be bounded within the range [0.08, 0.2] as specified in New Zealand design code SNZ 1995.

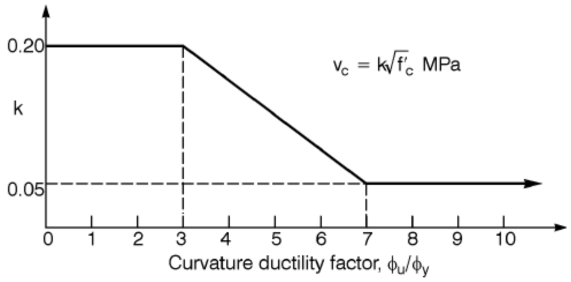
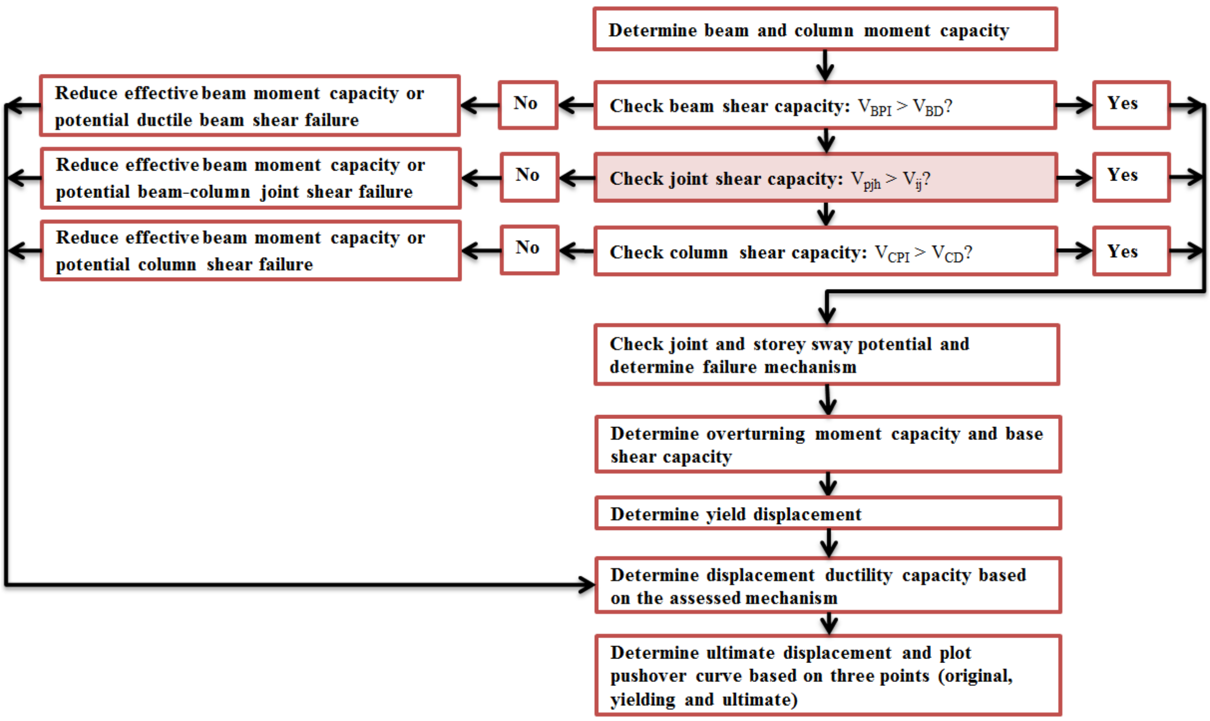


Figure 4- 3: Degradation of nominal shear stress resist by the concrete in beam

Beam shear demands should be estimated on the basis of the beam flexural capacities determined in the previous step, and if it is found that the shear demand of a beam exceeds the shear capacity (i.e. $V_{BPI} < V_{BD}$), a reduced effective beam moment capacity $M^*_{BI} = (V_{BPII} - V_{BGI})L_{bc} - M_{BNr}$ should be applied for this beam.

It is worth noting that the guidelines associated with the determination of beam shear capacity provided in NZSEE 2006 have not been updated according to the latest researches or the latest design code provisions regarding the estimation of beam shear capacity, or the latest experimental work related to the degradation of shear strength due to cyclic loading, etc. More discussions relating to the proposal to update the guidelines are shown in Chapter 5.

4.6.2.3. Check Beam-Column Joint Shear Capacity



Check if $V_{pjh}>V_{ij}$:

$$V_{pjh} = 0.85v_{ch}b_jh = 0.85k\sqrt{f_c'}\sqrt{1+\frac{N^*}{A_gk\sqrt{f_c'}}}b_jh \le 1.92\sqrt{f_c'}b_jh$$

V_{pjh}	=	Probable horizontal joint shear force
v_{ch}	=	Nominal horizontal joint shear stress carried by a diagonal compressive strut mechanism crossing joint
k	=	1.0 for interior joint 0.4 for exterior joint with beam longitudinal bars anchored by bending the hooks into the joint core 0.25 for exterior joint with beam longitudinal bars anchored by bending the hooks away from the joint core (into column above or below)
f_c'	=	Expected concrete compressive strength
b_j	=	Effective width of the joint (normally the column width)
h	=	Depth of column
A_g	=	Area of joint, $A_g=b_jh_c$

$$V_{ij} = \frac{\sum M_b}{0.9h_b} - \frac{\left(\sum M_b \frac{L_b}{L_{bc}}\right)}{L_c} \approx \sum M_b\left(\frac{1.1l_c - 1.2h_b}{h_bl_c}\right)$$

V_{ij}	=	Joint shear at moment capacities
$\sum M_b$	=	Summation of beam moment capacities at joint
h_b	=	Depth of beam
L_c	=	Column height (between beam centrelines)

As provided in NZSEE 2006, for the beam-column-joints without shear reinforcement at joint regions, shear capacities are estimated only considering concrete core contribution, and NZSEE 2006 does not provide any instructions to determine shear capacities of the joints with shear reinforcement. Axial loads N* are calculated by load combination N(G+ΨaQ) for interior columns (assuming insignificant effect of earthquake induced axial loads on the interior columns) and N(G+ΨaQ+E) for exterior columns. It is worth noting that E can be estimated as the sum of the exterior beam end shears. The value of k should be selected depending on joint types, and should be reduced due to cyclic lateral loading according to Figure 4- 4.

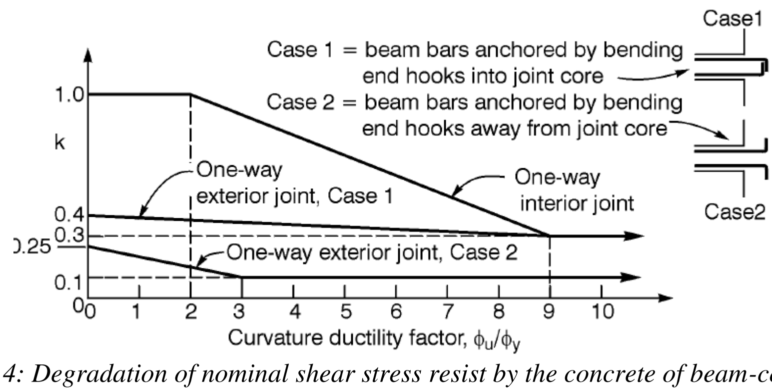


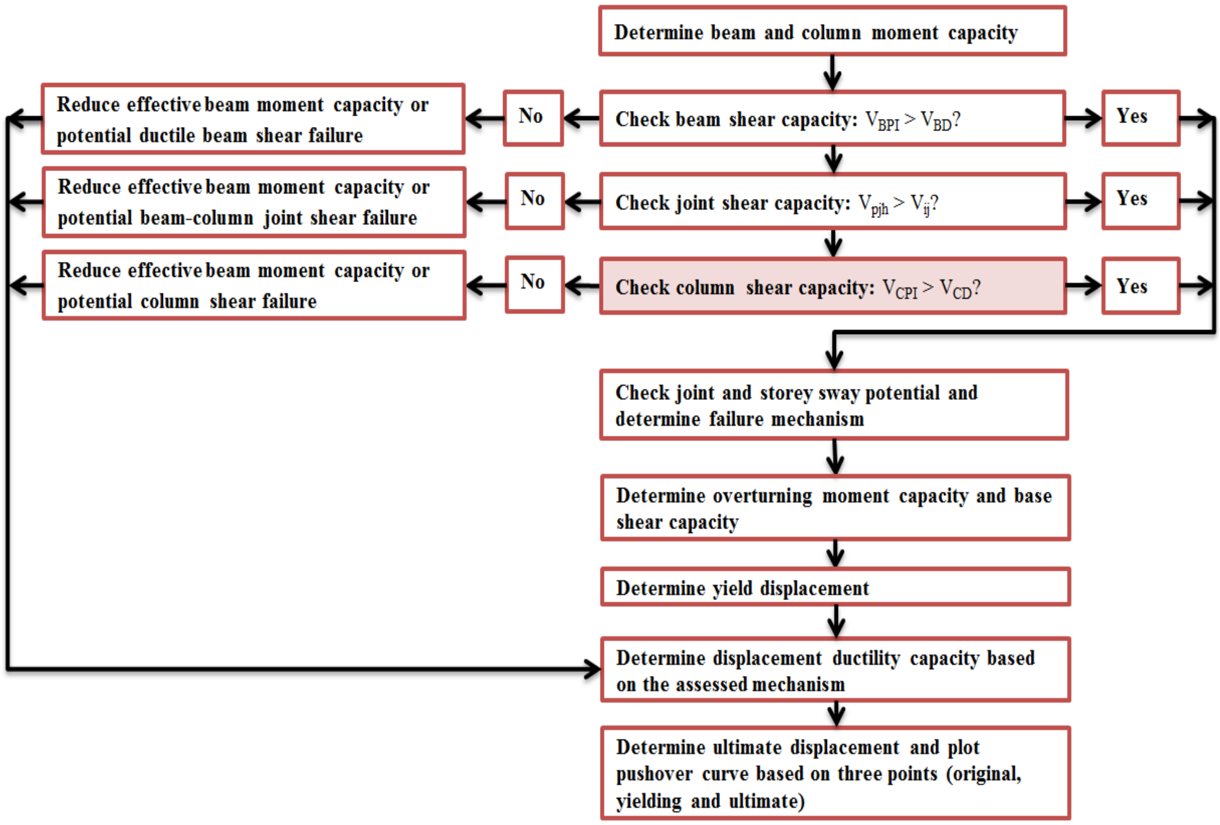
Figure 4- 4: Degradation of nominal shear stress resist by the concrete of beam-column joints

Beam-column-joint shear demands are estimated based on the beam flexural capacities determined in previous. If the shear demand of one joint is found to exceed the capacity, a reduced effective beam moment capacity should be determined as $\sum M_b = \frac{V_{ij}}{\frac{1}{0.9h_b}-\frac{\left(\frac{L_b}{L_{bc}}\right)}{L_c}} \approx V_{ij}\left[\frac{h_bl_c}{1.1l_c-1.2h_b}\right]$.

The following problems can be addressed during checking the beam-column joint capacity, and the suggestions to solve the problems are provided in Chapter 5.

- No detailed procedures are provided to calculate shear strengths of (1) the joints with sufficient shear reinforcement; (2) the joints with some but insufficient shear reinforcement.
- The guidelines associated with the determination of shear capacities of the joints without shear reinforcement may be out-of-dated.

4.6.2.4. Check Column Shear Capacity



Check $V_{CPI} > V_{CD}$:

$$V_{CPI} = 0.72(V_C + V_s + V_n)$$

V_{CPI}	=	Probable shear strength of columns (without plastic hinging)
V_C	=	Shear resisted by the concrete mechanisms
V_s	=	Shear resisted by the shear reinforcement (assuming that the critical diagonal tension crack is inclined at 30°)
V_n	=	Shear resisted as a result of the axial compressive load N*

$$V_C = v_c 0.8A_g = k\sqrt{f_c'}0.8A_g$$

v_c	=	Nominal shear stress carried by concrete mechanisms
k	=	$0.29\alpha\beta$ $1 \le \alpha = 3 - M/(VD) \le 1.5$ $\beta = 0.5 + 20\rho_l \le 1.0$ D = column diameter M/V = ratio of moment to shear at the section ρ_l = longitudinal column reinforcement ratio
A_g	=	Gross area of the column

$$V_s = \frac{A_v f_{yt} d''}{s} \cot 30^\circ \text{ for rectangular hoop}$$

$$V_s = \frac{\pi}{2} \frac{A_{sp} f_{yt} d''}{s} \cot 30^\circ \text{ for circular hoop}$$

A_v = Total effective area of hoops and cross ties in the direction of the shear force at spacing s

A_{sp} = Area of spiral or circular hoop bar

f_{yt} = Expected yield strength of the transverse reinforcement

d'' = Depth of the concrete core of the column measured in the direction of the shear force for rectangular hoops and the diameter of the concrete core for spirals or circular hoops

$$V_n = N^* \tan\infty$$

N* = Axial load on columns (seismic plus gravity)

∞ = For a cantilever column, ∞ is the angle between the longitudinal axis of the column and the straight line between the centroid of the column section at the top and the centroid of the concrete compression force of the column section at the base.
For a column with double curvature, ∞ is the angle between the longitudinal axis of the column and the straight line between the centroids of the concrete compressive forces of the column section at the top and bottom of the column.

$$V_{CD} = \omega_v \frac{M_{bijl} + M_{bijr} + M_{bi,j+1,l} + M_{bi,j+1,r}}{2kL_c} \leq \frac{M_{cijt} + M_{ci,j+1,b}}{kL_c}$$

V_{CD} = Column shear at moment capacities

M_{bijl} = Beam moment capacity at left of column i level j

M_{bijr} = Beam moment capacity at right of column i level j

$M_{bi,j+1,l}$ = Beam moment capacity at left of column i level j+1

$M_{bi,j+1,r}$ = Beam moment capacity at right of column i level j+1

M_{cijt} = Column moment capacity at top of column i level j

$M_{ci,j+1,b}$ = Column moment capacity at bottom of column i level j

k = (No specification)

ω_v = Dynamic magnification factor

$\omega_v = 0.9 + \frac{n}{10}$ if $n \leq 6$

$\omega_v = 1.3 + \frac{n}{30} \leq 1.8$ if $n > 6$

As provided in NZSEE 2006, column shear capacities are determined based on the shear resistance resulting from concrete mechanism, shear reinforcement and axial compressive load N*. The coefficient of 0.72 applied in the formula is derived from the multiple of a reduction factor (i.e. 0.85) and a modification factor (i.e. 0.85) based on experimental results. The k factor applied in the calculation of shear resisted by the concrete mechanism should be reduced due to cyclic lateral load, as shown in the following figure (Figure 4- 5).

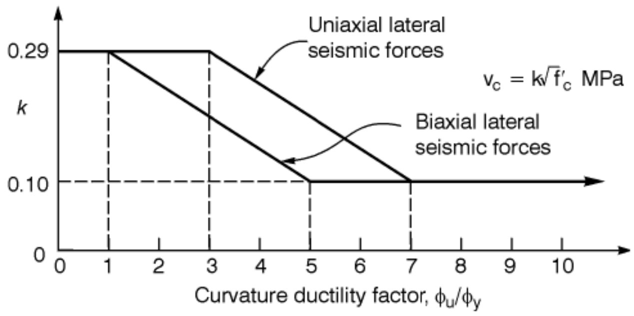


Figure 4- 5: Degradation of nominal shear stress resist by the concrete in column

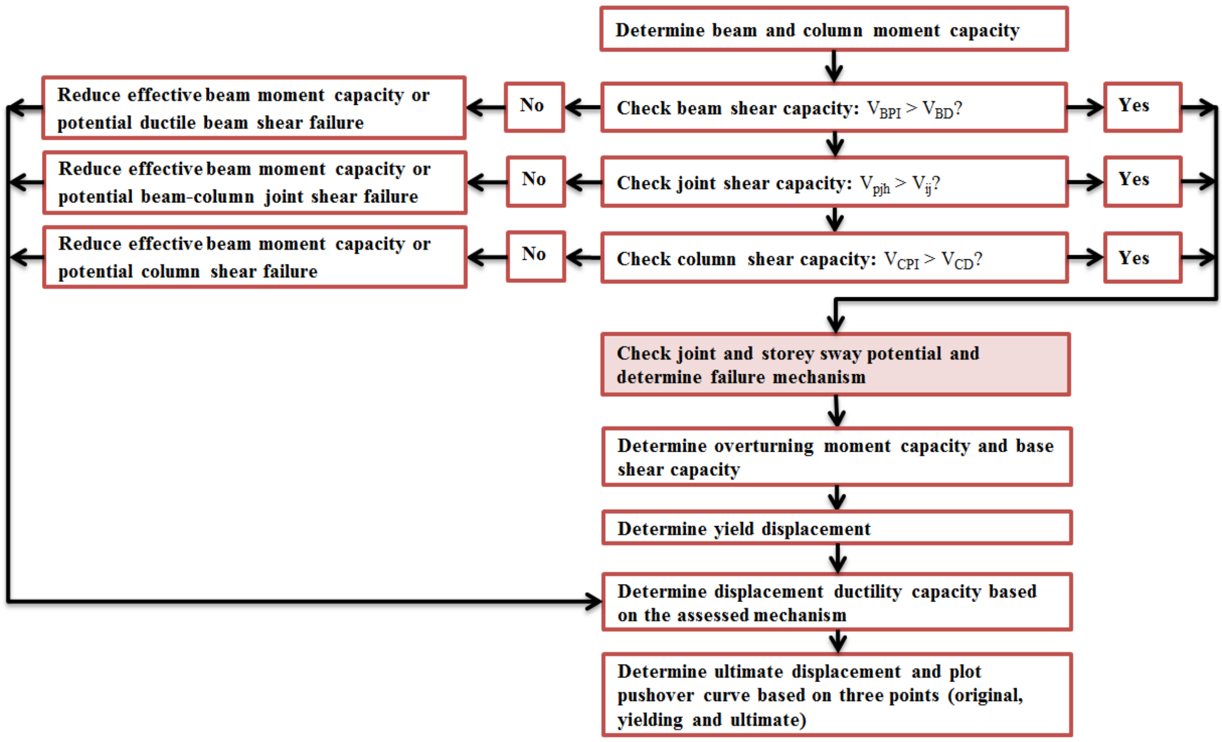
In calculating column shear demands, at ground level, the column flexural capacities are used, while for the levels above, the beam flexural capacities are applied. It is worth noting that a dynamic magnification factor, applied in order to account for higher mode effects, needs not to be considered if the storey sway potential S_i exceeds 0.85 (i.e. column plastic hinges are expected). An additional factor (i.e. k) is applied in determining V_{CD}, however, no definition of this factor is found in NZSEE 2006.

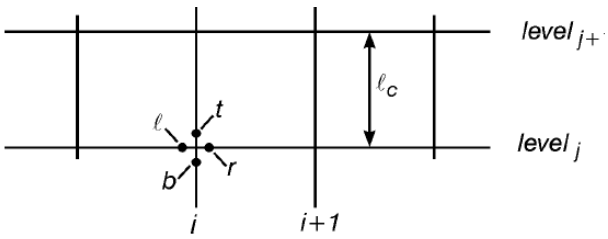
If it is found that the column shear demand of one column exceeds the capacity (i.e. $V_{CPI} < V_{CD}$), this column is expected to fail in a brittle manner, indicating that the displacement ductility capacity is low (e.g. 1). Thereby, beam moment capacities from the first step should be reduced depending on the calculated column shear demands.

The following problems can be addressed during checking column shear capacity, and the solutions to the problems are provided in Chapter 5.

- The guidelines provided in NZSEE 2006 to determine column shear capacity have not been updated with the most advanced researches, code provides or experimental work.
- There is a lack of sufficient instructions associated with the definition and determination of the dynamic magnification factor.
- There is a lack of guidelines provided regarding the specification of the k factor that is applied in determining column shear demands.

4.6.2.5. Check Joint and Storey Sway Potential and Determine Failure Mechanism





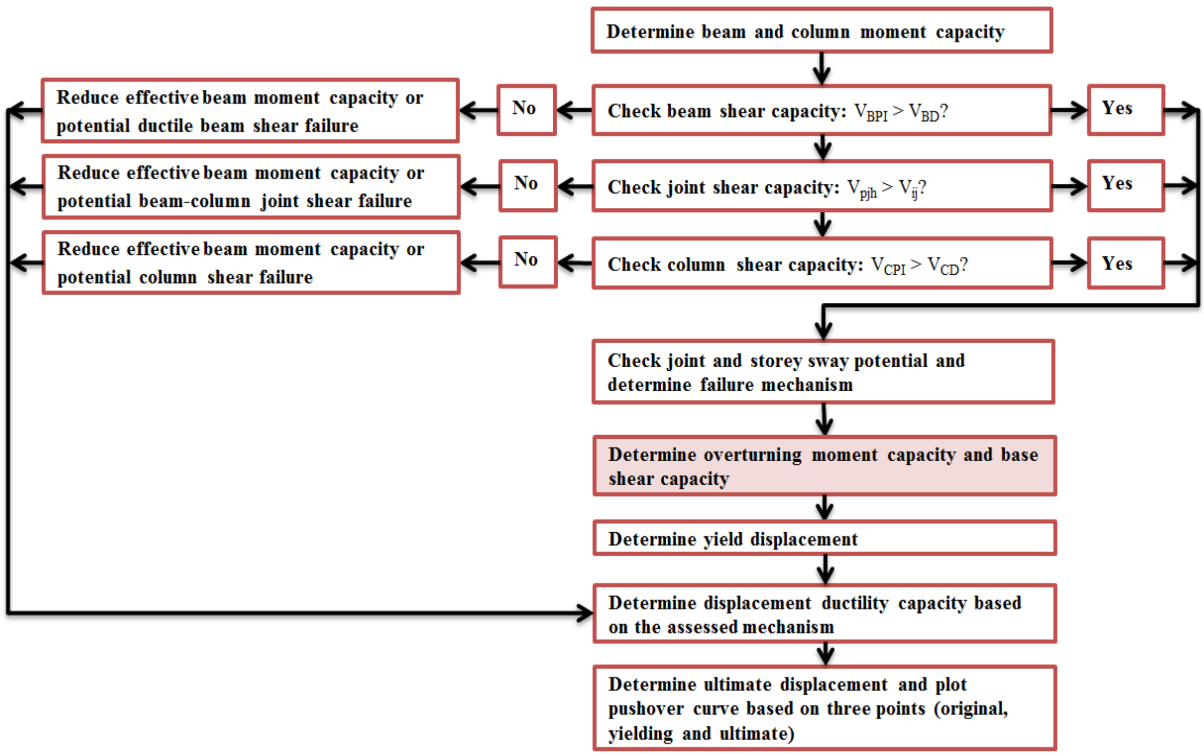
$$S_{Pij} = \frac{M_{bijl} + M_{bijr}}{M_{cijt} + M_{cijb}}$$

$$S^*_{Pj} = \frac{\sum_i (M_{bijkl} + M_{bijkr})}{\sum_i (M_{cijkt} + M_{cijkb})}$$

S_{Pij}	=	Sway potential at the joint on column I at level j
S^*_{Pj}	=	Sway potential at storey j
i	=	Column number
j	=	Storey number

If a joint sway potential (S_{Pij}) is found to be greater than 0.85, it can be expected that the column is "weaker" than the beam, in order words, column hinges form at top and/or bottom of the joint region before the formation of beam hinges. If the storey sway potential (S_{pj}*) is greater than 0.85 at the storey j, column side-sway mechanism is expected to occur at the storey j. It is worth noting that the application of the storey sway potential can lead to an overestimation of the probably lateral force capacity with a wrong failure mechanism captured (i.e. sway potential of each joint of storey j is not properly evaluated). A more robust procedure is required so that a correct failure mechanism can be predicted.

4.6.2.6. Determine Overturning Moment Capacity and Base Shear Capacity



$$OTM = \sum_i M_{coli} + N_E L$$

$$OTM_{Total} = \sum_k OTM_k$$

$$V_{base} = \frac{OTM}{h_{eff,one\ frame}}, V_{base,tolal} = \frac{OTM_{Total}}{h_{eff,whole\ structure}} \text{ or } \sum_k V_{base,k}$$

OTM	=	Overturning moment of a frame
$\sum_i M_{coli}$	=	Summation of column moment capacities at base
N_E	=	Axial force on column due to earthquake (estimated as the sum of the exterior beam shears)
L	=	Total span of the frame
OTM_{Total}	=	Total overturning moment of system (k frames)
V_B	=	Base shear capacity

$$h_{eff} = \frac{\sum m_j h_j^2}{\sum m_j h_j} \text{ (Force – based approach)}$$

$$h_{eff} = \frac{\sum m_i \Delta_i h_i}{\sum m_i \Delta_i} \text{ (Displacement – based approach)}$$

h_{eff}	=	Height of the lateral force resultant
m_j	=	Mass at storey j
h_j	=	Height of storey j

Alternatively, as defined in NZSEE 2006 7.2.4:

$$h_{eff} = 0.67H$$

Or for beam sidesway mechanisms:

$$h_{eff} = 0.64H, \text{for } n \leq 4$$

$$h_{eff} = [0.64 - 0.0125(n - 4)]H, \text{for } 4 < n \leq 20$$

$$h_{eff} = 0.44H, \text{for } n \geq 20$$

And for column sidesway mechanisms:

$$h_{eff} = 0.5H$$

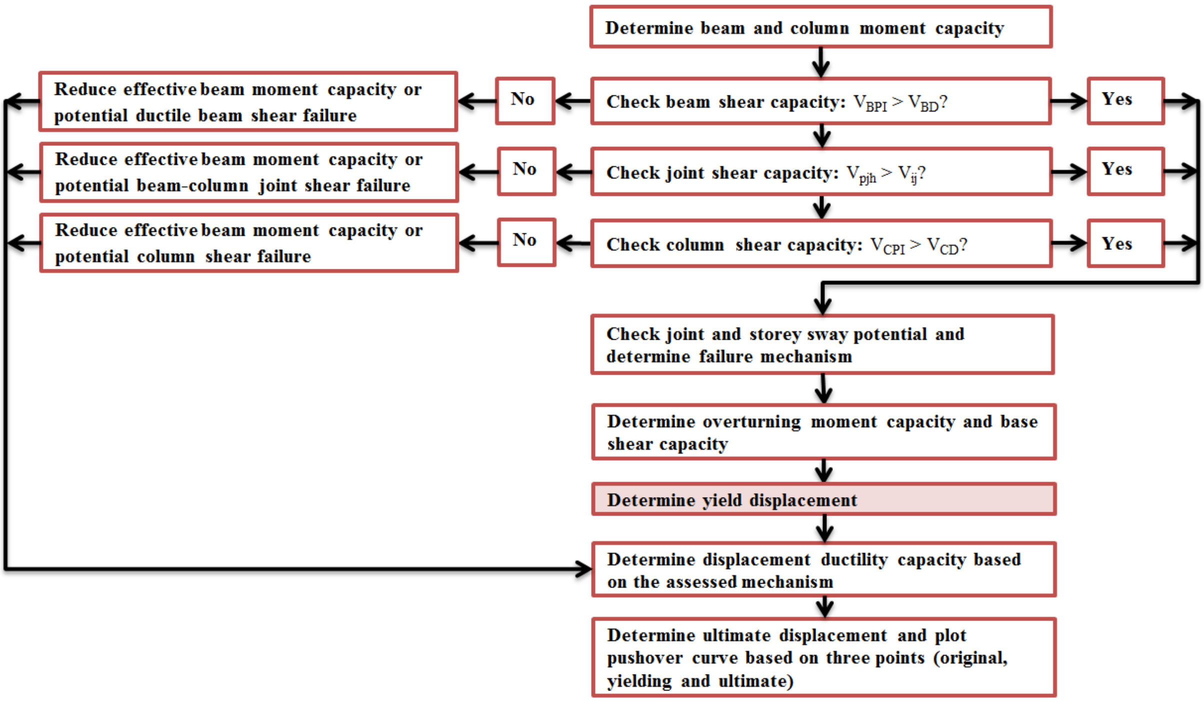
Or more accurately,

$$h_{eff} = \left(0.64 - 0.14\frac{\mu_s - 1}{\mu_s}\right)H, \text{and } \mu_s = 1 + \frac{(\varphi_u - \varphi_y)L_p H}{n\Delta_y}$$

The following problems can be addressed in the determination of overturning moment capacity and base shear capacity, and the measures to solve such problems are provided in Chapter 5.

- There is a lack of guidelines provided regarding the determination of structure displaced shape (Δi).
- There is a lack guidelines provided regarding the determination of total overturning moment for a structure with a column side-sway mechanism or a mixed side-sway mechanism.
- The guidelines provided in NZSEE 2006 to determine the effective height have not been updated with the most advanced researches.

4.6.2.7. Determine Yield Displacement

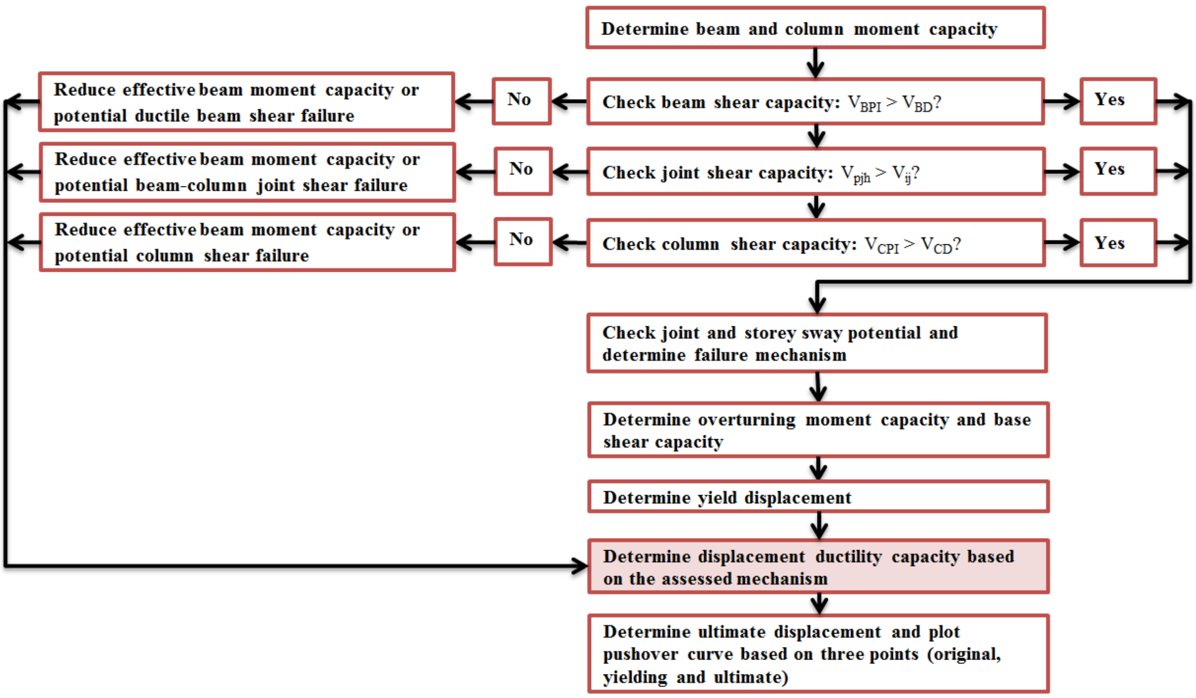


For beam side-sway mechanism:

$$\Delta_y = 0.5\varepsilon_y \frac{L_b}{h_b} h_{eff} \text{ or } [0.5\varepsilon_y \frac{L_b}{h_b} h_{eff}] \frac{OTM_1}{OTM_2}$$

h_{eff}	=	Height of the lateral force resultant
L_b	=	Full beam length
h_b	=	Beam depth
OTM_1	=	Unreduced beam moments
OTM_2	=	Beam moments reduced for ultimate joint shear
OTM_3	=	Beam moments reduced for the collapse mechanism

4.6.2.8. Determine Displacement Ductility Capacity of the Frame Based on the Assessed Mechanism



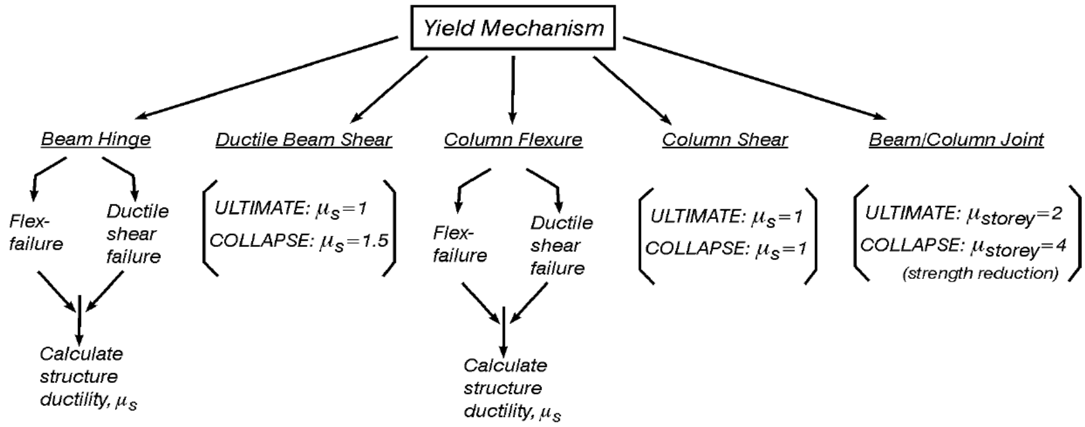


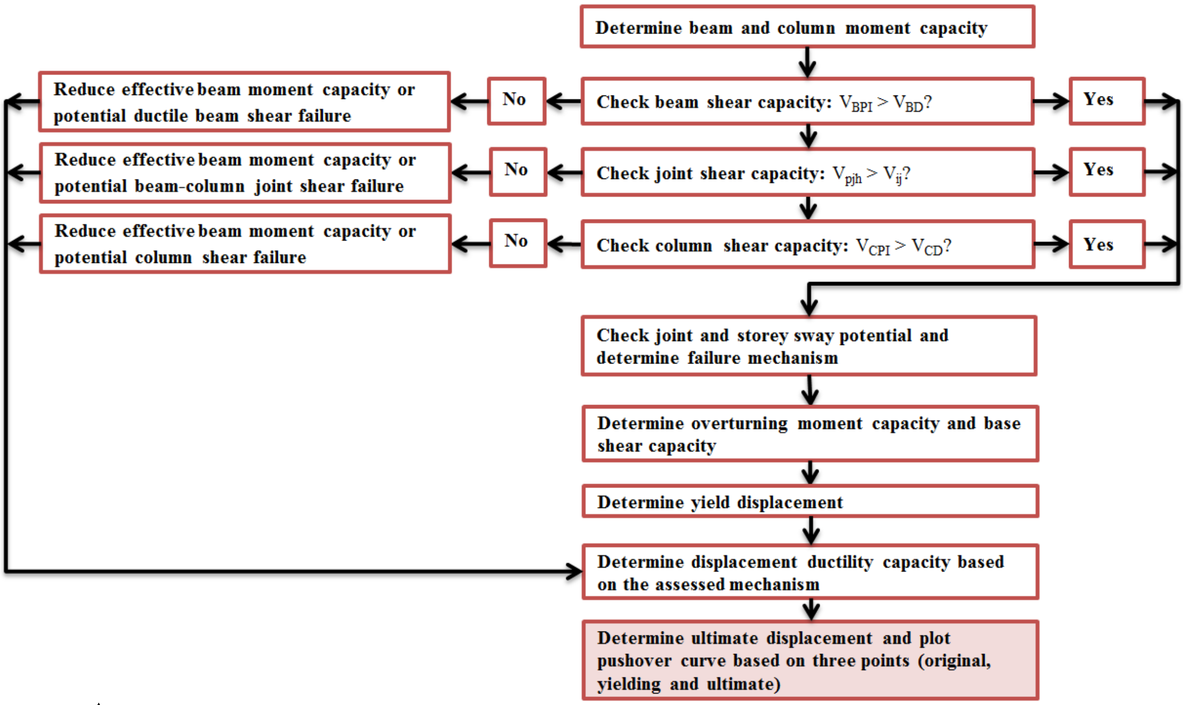
Figure 4- 6: Determination of frame ultimate displacement ductility capacity from NZSEE 2006 Appendix 4E

In Figure 4- 6, suggested values of ultimate displacement ductility capacity corresponding to different mechanisms are shown. In Table 4- 21, provided as a summary of guidelines associated with the determination of displacement ductility capacity, gather the information from NZSEE 2006 Appendix 4E (Section 10.3) and NZSEE 2006 Chapter 7 (Section 7.2.4.).

Table 4- 21: Suggested displacement ductility capacity corresponding to assessed mechanisms

Assessed Mechanism		**Suggested Displacement Ductility Capacity**
Column Bar Buckling	Premature column failure	Consideration should be given as to the likelihood of premature failure of this type occurring in columns with a high axial load and inadequate transverse reinforcement before undertaking numerical analysis on the more conventional possible failure modes.
Column Shear Failure	Inadequate column shear (prior to further mechanism forming)	Ultimate: $\mu_{sc} = 1$; Collapse: $\mu_{sc} = 1$
Beam Flexure Mechanism	Beam side-sway mechanism (flexure failure or ductile shear failure)	$L_{pb} = 0.08L + 0.022f_y d_b$ and $\theta_{pb} = (\varphi_u - \varphi_y)L_p$ L = Distance of the critical section and the point of contra-flexure, assumed to be $0.5L_b$ h_b = Beam depth n = Number of storeys φ_y and φ_u are determined from beam capacity calculation. For $n \leq 4$: $\mu_{sc} = 1 + \frac{0.64(\varphi_u - \varphi_y)L_p H}{\Delta_y}$ For $4 < n \leq 20$: $\mu_{sc} = 1 + \frac{[0.64 - 0.0125(n-4)](\varphi_u - \varphi_y)L_p H}{\Delta_y}$ For $n \geq 20$: $\mu_{sc} = 1 + \frac{0.44(\varphi_u - \varphi_y)L_p H}{\Delta_y}$
Beam Shear Failure	Inadequate beam shear	Ultimate: $\mu_{sc} = 1$; Collapse: $\mu_{sc} = 1.5$
Column Flexure Mechanism	Column side-sway mechanism (flexure failure or ductile shear failure)	$\mu_{sc} = 1 + \frac{(\varphi_u - \varphi_y)L_p H}{n\Delta_y}$ φ_y and φ_u are determined from column capacity calculation.
Beam Column Joint Shear Failure	Inadequate joint shear	Ultimate: $\mu_{storey} = 2$; Collapse: $\mu_{storey} = 4$

4.6.2.9. Determine Ultimate Displacement and Plot Pushover Curve



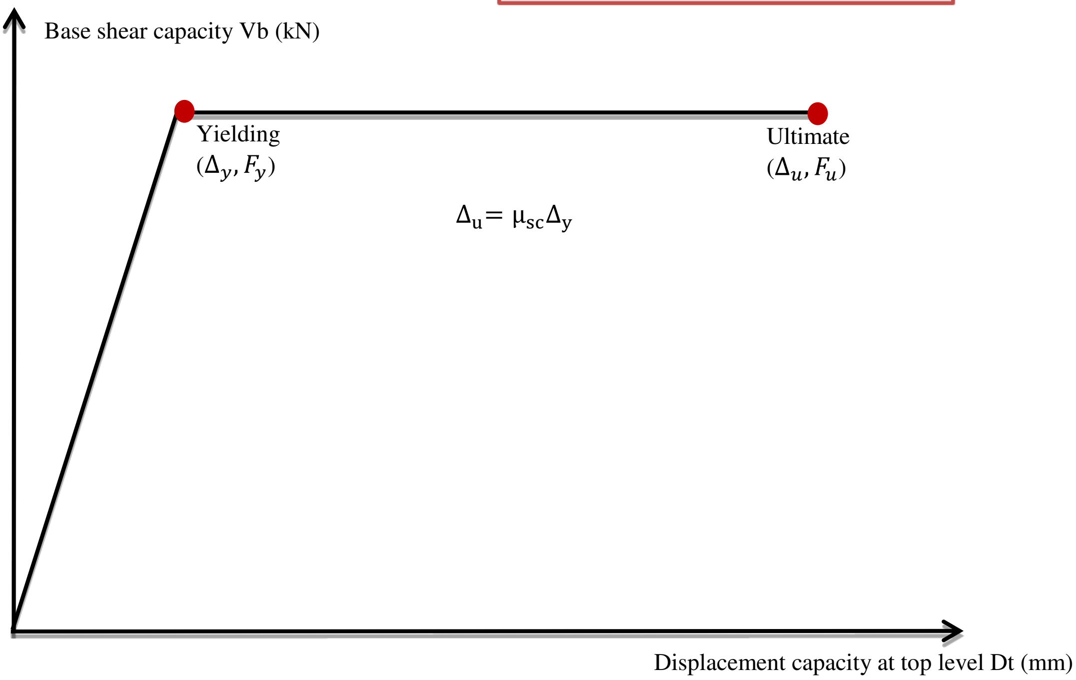


Figure 4- 7: Illustration of a bi-linear pushover curve

Based on the calculation in previous steps, a bi-linear pushover relationship can be computed, as illustrated in Figure 4- 7.

4.6.3. Limitations

The limitations of SLaMa are listed as following:

- The analysis is applicable only when there is no significant torsional stiffness irregularity.
- The analysis is applicable only when higher mode effects are not significant.
- The sequence of development of inelastic actions is not identified.
- The correct mechanism may be missed. For instance, there is possibility that even though flexural and shear checks for individual components are satisfied, and the storey does not have a sway potential, however, the actual response could be joint or column hinging before beam, which can actually triggering column sidesway mechanism. As a result, the lateral force capacity may be overestimated, especially for the structures with low member ductility capacity.
- Mixed sidesway mechanisms cannot be properly assessed.
- Secondary structural component and non-structural component cannot be accounted for in the analysis.
- The axial load contribution to the total overturning moment is not clearly defined.
- The determination of effective height not clearly defined, resulting in the Force-Based and the Displacement-Based procedures giving the same base shear capacity.

The problems addressed when performing a SLaMa to a frame manifest that the current SLaMa from NZSEE 2006 needs to be improved. The following chapter, Chapter 5, concentrates on the improvements suggested to the current SLaMa.

CHAPTER 5 Improvements to NZSEE 2006 (Focus on the Simplified Method, SLaMa)

5.1. Introduction

Based on the detailed study of NZSEE 2006 Guidelines and the critical comparison among the alternative assessment procedures (e.g. ASCE 41-13, EN 1998-3: 2005, NTC 2008, etc.), the deficiencies of NZSEE 2006 Guidelines can be identified, and the improvements can be suggested. Section 5.1 gives a summary of all the suggestions made, and the detailed suggestions regarding material level, component level, subassembly level and global level are discussed from Sections 5.2 to 5.5. It is worth noting that NZSEE 2006 Guidelines will be subject to ongoing refinement and development as further understandings and knowledge are gained.

- More detailed guidelines need to be provided concerning data collection for assessment:
  Based on the study of alternative assessment procedures, it can be suggested that different knowledge levels should be defined in New Zealand assessment guidelines (i.e. can be referred to Table 6-1 in ASCE 41-13 and modified to accommodate differences between New Zealand and America regulations). Future studies associated with the application of knowledge factors or confidence factors are required.
- More detailed recommendations need to be provided regarding the determination of material properties and strengths, component properties and strengths (including secondary structural and nonstructural components). The recommendations should refer to the following resources of information.
  - i. Construction documents, structural drawings, survey data, on-site investigations, physical testing, etc
  - ii. The past and the current New Zealand design standards, i.e. design history
  - iii. The most advanced knowledge acquired from the latest researches and experimental work
  - iv. Knowledge, along with suggestions and instructions, from the alternative assessment standards or guidelines
- The analysis approaches should be improved: Equivalent Static Analysis (i.e. LSP). Modal Response Spectrum Analysis (i.e. LDP), Lateral Pushover Analysis (i.e. NDP), and Inelastic Time History Analysis (i.e. NDP) should be improved based on the most advanced knowledge from the latest researches and from alternative assessment procedures.
  As for the current simplified analytical approach, i.e. SLaMa, the following modifications or improvements are suggested, with details shown from Section 5.4 to 5.7.

i. Modifications of the formulae (including coefficients) applied to estimate effective height, displaced-shape, etc

ii. Evaluation of strength hierarchy

iii. Determination of lower and upper bounds of lateral load capacity

iv. Determination of sequence of mechanisms by Portal Frame Method

v. Adoption of component analysis model and global structure model

vi. Application of SLaMa approach to shear wall structures

- While it is generally considered that the displacement-based procedure produce more rational and less conservative outcomes, most designers are currently more familiar with the force-based procedure. Therefore, the displacement-based procedure should be well-explained in the Guidelines and should be encouraged to apply in practice. It can also be encouraged that both procedures are performed in practice, and the outcomes of one procedure can be cross-checked by the outcomes of the other.
- The determination of demand should be improved, e.g. the application of ADRS format for seismic demand should be encouraged. More details regarding this aspect are discussed in Chapter 7 and Chapter 9.

5.2. Material Level

As discussed in Section 3.3.3, even though instructions to obtain material properties and strengths are provided in the current Guidelines, there is a lack of instructions to approximate material properties and strengths in the absence of information. Therefore, it can be suggested that more clarified requirements for data collection should be defined, and more specified instructions to obtain material properties corresponding to the quantity and quality of data collected should be included. Under the circumstance of only limited information being available, the followings can be suggested and should be included in the Guidelines:

- Material properties and strengths can be estimated on the basis of the history of material properties and strengths concluded from the available building construction documents, structural drawings, surveys data on-site investigations, physical testing data, etc. of some building representatives For the buildings without the access to sufficient information, it can be deduced that they should have the similar material properties and strengths with the buildings of the same design and construction period. Table 6- 10 and Table 6- 12 in Section 6.3.3.1.1 and Section 6.3.3.2.1 provide summaries of concrete compressive strength and reinforcing steel yield strength applied in different periods of time according to the information collected for a few building representatives (i.e. the 22 RC buildings of Knowledge Level 2 in the Refined Database). Therefore, the material properties and strengths of the buildings under assessment can be approximated by selecting proper building representatives from Table 6- 10 and Table 6- 12. However, the reliability of these tables should be improved by including more building representatives. This requires future data collection work.

- Material properties and strengths can be estimated on the basis of the history of material properties and strengths applied in New Zealand past and current designs. Table 6- 11 and Table 6- 13 in Section 6.3.3.1.2 and Section 6.3.3.2.2 provide summaries of concrete compressive strength and reinforcing steel yield strengths from New Zealand design standards. It is worth noting that apart from the suggested values from the design standards, the recommendations or suggestions that may affect the determination of material properties and strengths also need to be accounted for. More detailed information regarding the material properties and strengths is tabulated in Appendix A7. These tables were prepared by SAFER group members.
- Material properties and strengths can be estimated on the basis of the most advanced knowledge from the latest research and experimental work. SAFER research group members have already carried out researches associated with old concrete hardening issues, steel hardening issues, plain steel bar issues, degradation of strength, residual strength, material overstrength factors, defining limit states at material level, and so on. The Guidelines should be subject to ongoing improvements as further research outcomes are gained.
- Material properties and strengths can be estimated on the basis of the knowledge, along with suggestions and instructions from the alternative assessment standards or guidelines.
    i. The definition of knowledge level from ASCE 41-13 or EN 1998-3: 2005 (or NTC 2008) can be adopted to establish requirements for data collection.
    ii. The table format of design history of material properties and strengths together with the recommendation of default values from ASCE 41-13 can be introduced in NZSEE.

  More details are shown in Section 3.3.3.1 and Section 3.3.3.2, Table 3- 23 and Table 3- 28.
- The impact on the final assessment results due to the variation of material properties and strengths should be specified. The use of material nominal strength, probable strength and overstrength should be clarified, and proper material strength variation ranges should be suggested corresponding to the different requirements of sophistication. More discussions regarding this aspect are shown in Chapter 7 and Chapter 9.

5.3. Component Level

The current guidelines provide some specifications of component properties and instructions to determine component strengths. However, due to the issues addressed in Section 4.6, improvements to the guidelines are necessary. In the current guidelines, the instructions to determine beam flexural and shear strength, column flexural and shear strength, joint shear strength, and wall flexural and shear capacity are based on New Zealand design standard SNZ 1995 (i.e. NZS3101:1995), which has already been superseded by NZS3101:2006. Hence, these instructions, including formulae, values of coefficient, etc., should be modified referring to NZS3101:2006 and the latest researches. In addition, the guidelines should also be improved by comparing to the alternative assessment standards or guidelines. In Section 3.3.4, the differences in the determination of component strengths, presentation of component capacities, acceptance criteria, etc. in the four codified assessment procedures are shown.

Details of suggestions for each type of components (excluding structural walls) are discussed from Section 5.3.1 to 5.3.3.

5.3.1. Beam

The detailed procedure to determine beam flexural and shear strength from the current guidelines is shown in Section 3.3.4.1 and Section 4.6, and the followings are suggested in aiming to improve the guidelines.

- The accuracy of the estimation of beam strengths needs to be specified according to different levels of evaluation. For instance, in a quick evaluation, a preliminary approximation of beam flexural strength is sufficient, however, in a more detailed evaluation, more sophisticated section analyses either by hand or computer program are required to apply. It should be clarified that bilinear, tri-linear, or even more complicated moment-curvature relationships or force-deformation relationships should be assumed according to different levels of evaluation.
- The formulae applied, along with the suggested values of some coefficients or parameters (e.g. k value that is associated with concrete shear stress, overstrength factor, etc.), to determine beam flexural and shear strengths should be updated according to the latest design standard, e.g. NZS3101:2006, or should be updated according to the latest researches.
- The information associated with slab contribution to beam flexural strength, bond deterioration, bond slip effect, beam overstrength, strength degradation, residual strength, etc. should be updated with the most advanced knowledge from the latest researches.
- The knowledge, together with the recommendations or instructions from the alternative assessment standards or guidelines (i.e. see Table 3- 34 from Section 3.3.4.1) may be adopted.
    i. Component force-deformation analysis models from ASCE 41-13 may be adopted in NZSEE. It is worth noting that such force-deformation beam models can be refined by interpreting the characteristics such as strength degradation, residual strength, and limit state criteria.

ii. The suggested acceptance criteria and limit state criteria for beam components from ASCE 41-13 may be adopted in NZSEE.

- The impact on the final assessment results due to the variation of the calculated beam strength should be specified. The use of beam nominal strength, probable strength and overstrength should be clarified. Proper beam strength variation ranges may be defined, consistent with the required level of sophistication. More discussions regarding this issue are shown in Chapter 7 and Chapter 9.

5.3.2. Column

The detailed procedure to determine column flexural and shear strength from the current guidelines is shown in Section 3.3.4.2 and Section 4.6. As mentioned in Section 4.6, there is a lack of specifications regarding the interaction between column strengths and imposed axial loads, and some other aspects such as the characteristics of circular sections, etc. The followings can be suggested:

- The procedure to conduct moment-axial load interaction analysis should be included in NZSEE, then the impact of axial load on flexural capacity of column can be directly visualised from the moment-axial load interaction curve as shown in Figure 5- 1. The detailed calculation process of the moment-axial load interaction analysis is also presented in Figure 5- 1. Apart from performing a moment-axial load interaction analysis, the impact of the imposed axial load on column curvatures needs to be accounted for. For simplification purposes, compared to the influence of the axial load on ultimate curvature, the influence of the axial load on yield curvature is insignificant and can be neglected.

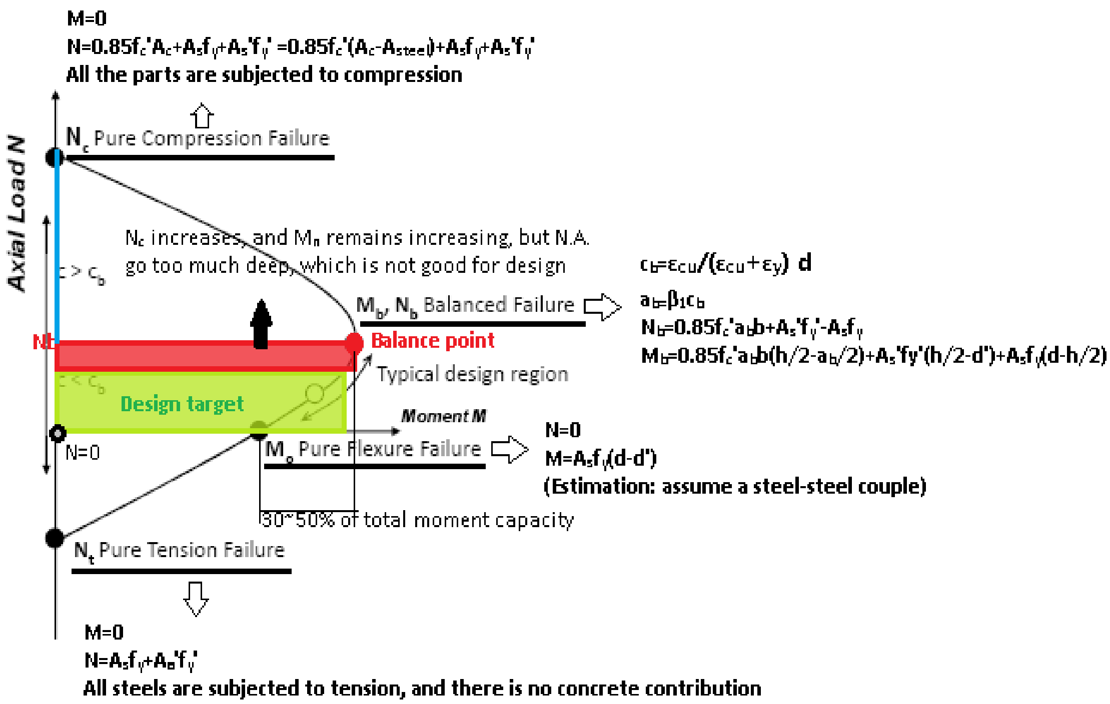


Figure 5- 1: Summary of moment-axial load interaction analysis procedures for column

- In determining the shear strengths for a column, the N* value should be estimated from the load combination of seismic induced axial force and gravity (dead load plus live load), as specified in NZSEE2006 Appendix 4E.10.1.8. It can be argued that for simplification purposes, in a frame structure with multiple bays, the effect of the earthquake induced axial loads on the interior columns can be ignored, and it can be assumed that the earthquake axial loads are only resisted by the exterior columns. Hence, $N^*=G+\Psi_cQ+E$ (where $\Psi_c=0.4$ from NZS1170.0 Table 4.1 Ultimate Limit State) should be applied in the calculation for the exterior columns, while $N^*=G+\Psi_cQ$ ($\Psi_c=0.4$) for the interior columns. Special attention should be arisen when neglecting the impact of earthquake induced axial loads on the interior columns in calculation process, as the applicability of such simplification depends on level of evaluation required. More refined calculation of earthquake induced axial forces is required in a sophisticated assessment.
- The earthquake induced axial load, i.e. E, can be estimated as the sum of exterior beam end shear capacities and factored by R_V, where R_V is the axial load reduction factor and should be calculated by $R_V = 1.0 - 0.015n \geq 0.70$ (n is the number of storeys). Hence, the earthquake induced axial load should be calculated as $N_{OE} = R_V \times \sum V_{Ob}$, where $\sum V_{Ob}$ is the sum of shears in beams due to the end moments which are sustained when overstrength actions exist in the beams. It is worth noting that R_V was not taken into consideration during calculation of the case studies where nominal or probable material and component strengths were applied, since the factor R_V arises from the overstrength of beams.
- Similar to the suggestion proposed for beams, the accuracy of the estimation of column strengths may need to be defined according to different levels of evaluation.
- Also similar to the suggestion proposed for beams, the formulae applied, along with the suggested values for some coefficients or parameters (e.g. k value that is associated with concrete shear stress, overstrength factor, etc.), to determine column flexural or shear strengths should be updated according to the latest design standard, e.g. NZS3101: 2006, or should be updated according the latest researches.
- Depending on the level of evaluation carried out, assumptions should be allowed in order to simplify the calculation process. For instance, during calculating column shear strength, it may be assumed that the shear resisted as a result of the axial compressive load (Vn) can be ignored when Vc and Vs dominate the contribution to shear resistance. Otherwise, the angle ∞ should be assumed as zero initially and revised after displacement at top of the structure being calculated out (i.e. $\tan\infty \approx \Delta u/H$). For another example, assumption of $\alpha = 1$ (i.e. simplification of calculating k value) may be adopted if there is lack of information regarding the concrete mechanism to resist shear.

- The information associated with bond deterioration, bond slip effect, strength degradation, residual strength, etc. should be updated with the most advanced knowledge from the latest researches.
- The knowledge, together with the recommendations or instructions from the alternative assessment standards or guidelines (i.e. see Table 3- 37 from Section 3.3.4.2) may be adopted.
  i. Component force-deformation analysis models from ASCE 41-13 may be adopted in NZSEE. It is worth noting that such force-deformation column models can be sophisticated by interpreting the characteristics such as strength degradation, residual strength, and limit state criteria.
  ii. The suggested acceptance criteria and limit state criteria for column components from ASCE 41-13 may be adopted in NZSEE.
- The impact on the final assessment results due to the variation of the calculated column strength should be specified. The use of column nominal strength, probable strength and overstrength should be clarified. Proper column strength variation ranges may be defined, consistent with the required level of sophistication. More discussions regarding this issue are shown in Chapter 7 and Chapter 9.
- Procedures to estimate flexural and shear strength of circular columns should be included. One of the measures that can be applied is to assume an equivalent square section profile for a circular column section, while this method may not provide as good approximation that is required in assessment. Therefore, section analysis computer programs, for instance, Response 2000, ABSTRACT, etc., are recommended. It should be noted that whether to apply the assumed section in quick hand calculation or to apply computer analysis depends on the requirements of sophistication.
- As mentioned in Section 4.6.2.4, during the calculation of column shear demands, more clarified guidelines should be included. For instance, there is lack of information regarding the definition of dynamic magnification factors. Table 5- 1 summarises the specifications associated with dynamic magnification from NZS3101: 1995 (i.e. same in NZS3101:2006) and PRESSS Design Book, and they are not consistent with each other. Hence, further investigation in this is required.

Table 5- 1: Specifications of dynamic magnification from NZS3101: 1995 and PRESSS Design Book

NZS3101: 1995	PRESSS Design Book
$\omega_v = 0.9 + \frac{n}{10}$ if $n \leq 6$ $\omega_v = 1.3 + \frac{n}{30} \leq 1.8$ if $n > 6$ Where n is the number of building levels	$\omega_v = 1.0$ if $H_n < 45m$ $\omega_v = 1.15 - 0.0034H_n$ if $H_n > 45m$ Where Hn is the total height of the building (from ground to roof)

5.3.3. Joint

The detailed procedure to determine shear strength of the joint without shear reinforcement from the current guidelines is shown in Section 3.3.4.3 and Section 4.6. As stated in Section 4.6.2.3, the current guidelines lack instructions concerning estimating the shear strength of the joint with shear reinforcement. Thus, the followings can be suggested:

- The procedure to determine joint capacity in terms of column moment (i.e. joint equivalent moment) should be included, so that the joint capacity versus axial load can be plot in the same domain of column moment-axial load relationship. The details of the calculation process are presented at the end of this section.
- Similar to the suggestions proposed for beams and columns, the formulae applied, along with the suggested values for some coefficients or parameters (e.g. k value that is associated with concrete shear stress), to determine joint capacities should be updated according to the latest design standard, e.g. NZS3101: 2006, or should be updated according to the most advanced knowledge from the latest researches. In this section, a brief summary of researches in strength degradation of joint in the past twenty years is included, as shown in Figure 5- 3, Figure 5- 4 and Figure 5- 5.
- Instructions should be included to determine the capacities of the joints with sufficient shear reinforcement or the joints with some but insufficient shear reinforcement. The capacities of such joints should be calculated from the sum of concrete and shear reinforcement contributions. *Tasligedik (2014)* suggested that, the horizontal shear stress at joint with shear reinforcement can be calculated as:

$$\tau_{jt} = \frac{V_{jt}}{b_c d_c} = \frac{\left(1 + \frac{K \cdot N_{cj}}{b_c h_c f'_c}\right) v_b b_c d_c + \frac{A_v}{s} f_y \cdot d_c}{b_c d_c} \text{ in MPa}$$

$$0.08\sqrt{f'_c} \le v_b = (0.07 + 10 \cdot \frac{\sum A_{stc}}{b_c d_c}) \cdot \sqrt{f'_c} \le 0.2\sqrt{f'_c} \text{ in MPa}$$

K	=	3 for columns in compression, i.e. $N_{cj} > 0$ 12 for columns in tension, i.e. $N_{cj} < 0$ 0 for columns with zero axial load, i.e. $N_{cj} = 0$
d_c	=	Effective depth of the column
A_v	=	Total area of the shear reinforcement in the joint (i.e. total of the legs)
f_y	=	Yield strength of the shear reinforcement
f_c’	=	Concrete compressive strength
A_{stc}	=	Total area of the longitudinal steel reinforcement in the column (accounts for the dowel action of the rebar)

It is worth noting that the calculation is based on design standard NZS3101:2006, which should be subject to ongoing refinement and improvement as future researches and investigations are carried out.

- The knowledge, together with the recommendations or instructions from the alternative assessment standards or guidelines (e.g. see Table 3- 42 from Section 3.3.4.3) may be adopted, e.g. the joint analysis model with numerical acceptance criteria from ASCE41-13.

<u>Joint shear stress strain relationships (considering only concrete mechanism contribution) based on experimental researches:</u>

In the current NZSEE 2006 guidelines, k values (from the $p_t=k\sqrt{f_c'}$) are specified under three circumstances, (1) interior joint; (2) exterior joint with beam longitudinal bars anchored by bending the hooks in to the joint core; (3) exterior joint with beam longitudinal bars anchored by bending the hooks away from the joint core (into column above or below), illustrated in Figure 5- 2.

K = 2.0 for interior joint

0.4 for exterior joint with beam longitudinal bars anchored by bending the hooks into the joint core

0.25 for exterior joint with beam longitudinal bars anchored by bending the hooks away from the joint core (into column above or below)

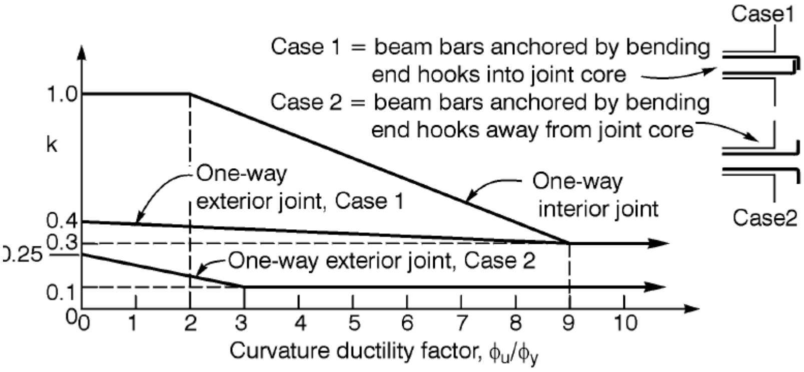


Figure 5- 2: Degradation of nominal shear stress resist by the concrete of beam-column joints (NZSEE 2006)

Figure 5- 3, Figure 5- 4 and Figure 5- 5 illustrate joint shear stress strain models from several researches in history.

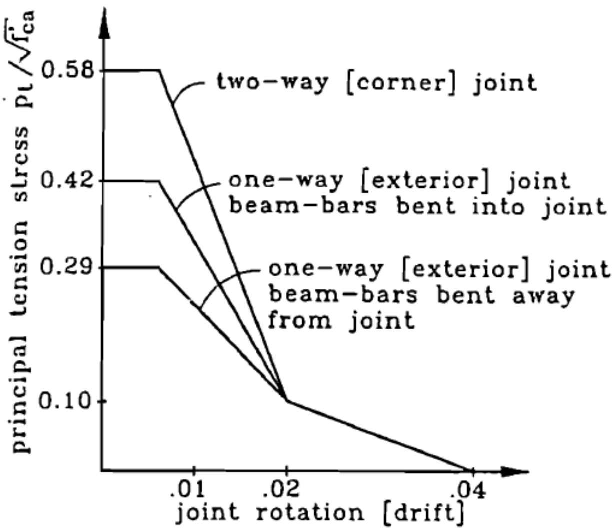


Figure 5- 3: Suggested strength degradation model for exterior and corner joints (Priestley 1997)

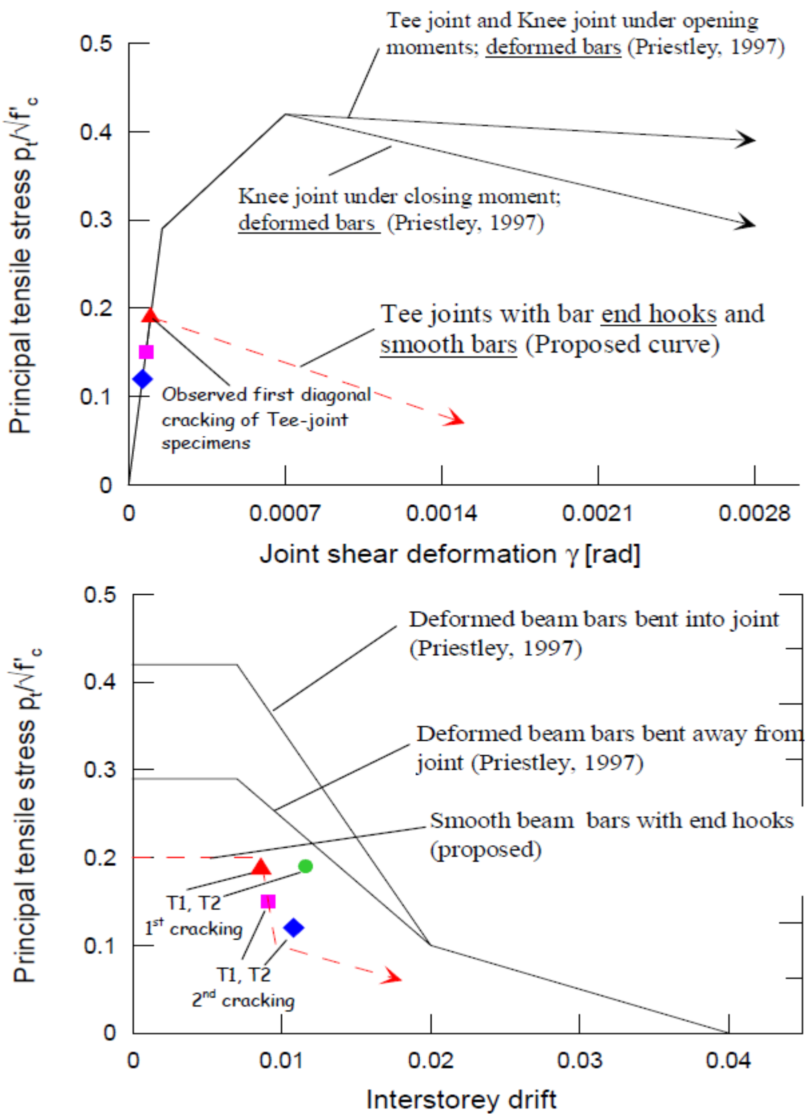


Figure 5- 4: Strength degradation curves for exterior joints (S. Pampanin. 2002)

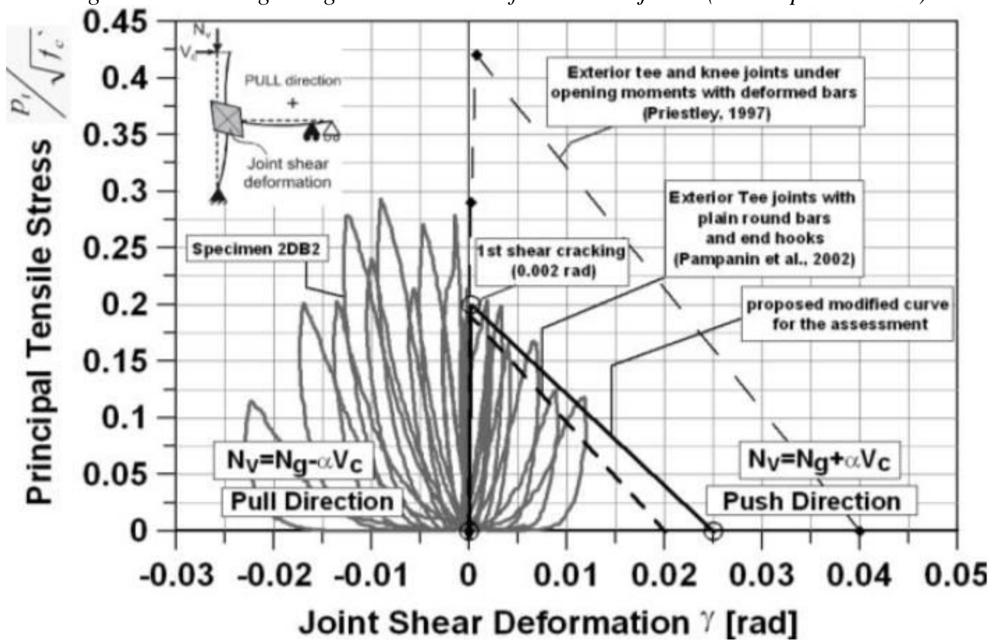


Figure 5- 5: Strength degradation curves for exterior joints in terms of principal tensile stress vs. joint shear deformation (U. Akguzel. 2012)

Procedure to calculate capacity for an exterior joint: (based on Mohr's Circle Theory)

Horizontal shear force acting on the joint core: $V_{jh} = T - V_c$

Equilibrium of the external action: $V_c l_c = V_b l_b$

Rearrange to get V_b: $V_b = \frac{V_c l_c}{l_b}$

Moment acting at the face of the joint core: $M_b = V_b\left(l_b - \frac{h_c}{2}\right) = Tjd$

Rearrange to get T:
$$T = \frac{M_b}{jd} = \frac{V_b\left(l_b - \frac{h_c}{2}\right)}{jd} = \frac{V_c l_c\left(l_b - \frac{h_c}{2}\right)}{l_b jd}$$

Substitute to the 1st equation:
$$V_{jh} = T - V_c = \frac{V_c l_c\left(l_b - \frac{h_c}{2}\right)}{l_b jd} - V_c = V_c\left[\frac{l_c}{l_b jd}\left(l_b - \frac{h_c}{2}\right) - 1\right]$$

Rearrange to get V_c:
$$V_c = \frac{V_{jh}}{\left[\frac{l_c}{l_b jd}\left(l_b - \frac{h_c}{2}\right) - 1\right]}$$

Joint capacity in terms of column moment:
$$M_{col} = V_c\left(\frac{l_c - h_b}{2}\right) = \frac{V_{jh}}{\left[\frac{l_c}{l_b jd}\left(l_b - \frac{h_c}{2}\right) - 1\right]}\left(\frac{l_c - h_b}{2}\right)$$

Assume j=0.9d and $A_e = b_j \times h_c$:
$$M_{col} = \frac{\nu_{jh}(1000)}{\phi}\ \text{kNm and } \phi = \frac{2l_b' l_c - 1.8dl_b}{0.9dl_b A_e(l_c - h_b)}$$

Nominal horizontal shear stress at the mid-depth of the joint core:
$$\nu_{jh} = \frac{V_{jh}}{b_j \times h_c}$$

Effective width of the joint:
$$b_j = \min(b_c, b_w + 0.5h_c)\ \text{if } b_c \geq b_w$$
$$b_j = \min(b_w, b_c + 0.5h_c)\ \text{if } b_c \leq b_w$$

Principal tensile and compressive stresses:
$$p_{t,c} = -\frac{f_v}{2} \pm R$$

Substitute $R = \sqrt{\left(\frac{f_v}{2}\right)^2 + \nu_{jh}{}^2}$ from Mohr's Circle Theory:
$$p_t = -\frac{f_v}{2} + \sqrt{\left(\frac{f_v}{2}\right)^2 + \nu_{jh}{}^2}$$

Rearrange to get horizontal shear:
$$\nu_{jh} = \sqrt{p_t{}^2 + p_t f_v}$$

Substitute to the joint capacity:
$$M_{col} = \frac{\sqrt{p_t{}^2 + p_t f_v}(1000)}{\phi}\ \text{kNm}$$

Principal tensile stress:
$$p_t = k\sqrt{f_c'}$$

Stress due to axial load:
$$f_v = \frac{N_v}{A_e}$$

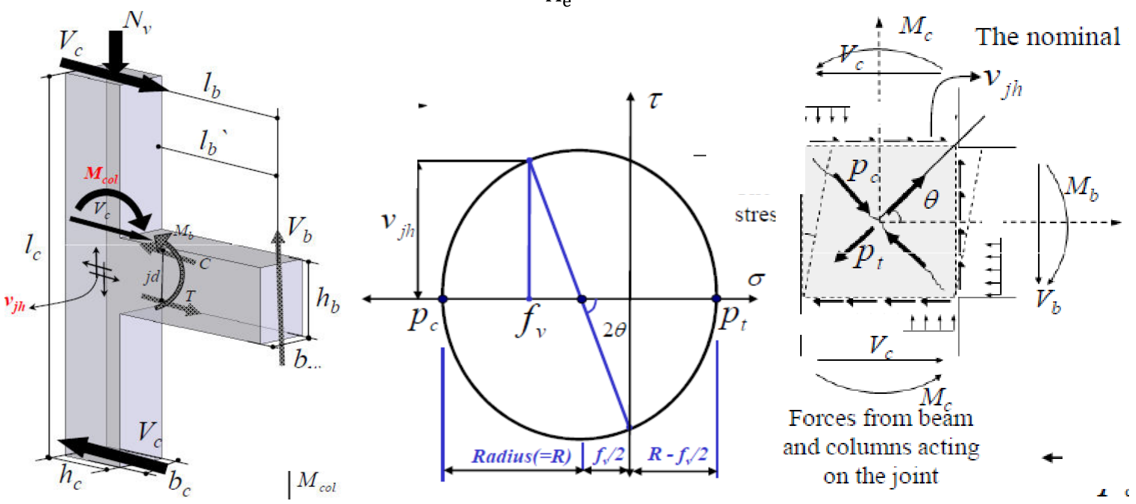
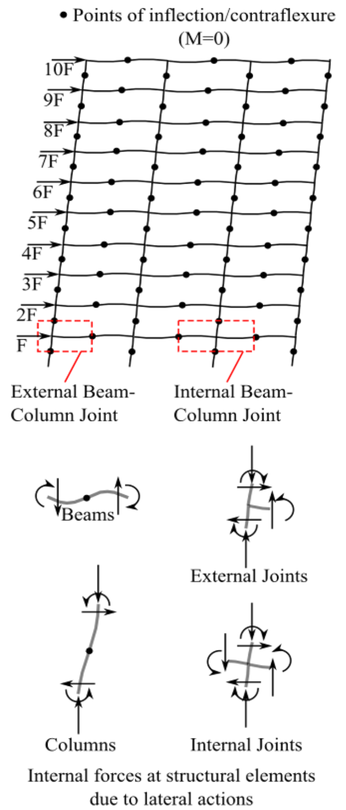
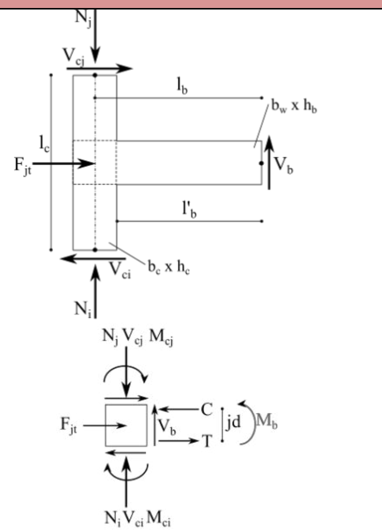
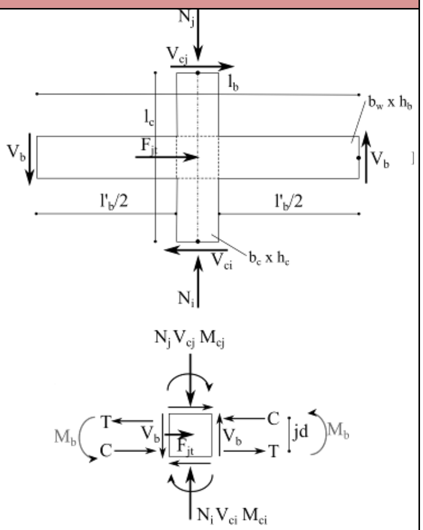


Figure 5- 6: Left: Free-body diagram of as-built specimen; Middle: Mohr's Circle Theory applied to calculate joint shear (Principal Tensile Stress Approach); Right: Illustration of stress, shear and moment at joint region

For the interior joints, the same calculation process should be followed, but minor changes should be made, for instance, the l_b' and l_b should be the beam clear span and full span.

On the basis of the procedure shown in the previous page, *Tasligedik (2014)*suggested that the shear capacity of a joint can be determined in terms of the column moment, the axial load on the column, and the effect of the equivalent static force fraction (F_{jt}) on the joint, as shown in Table 5- 2. As explained in the *Tasligedik*'s paper *"Calculation of Strength Hierarchy at Reinforced Concrete Beam Column Joints: from Experimental Studies into Structural Engineering Applications"*, the consideration of F_{jt} makes the strength hierarchy procedure applicable for the entire structure, hence, the sequence of mechanisms at global structure level can be predicted. However, it is worth noting that the method cannot compute pushover curves, in another word, the method cannot provide estimation of lateral load capacity nor displacement capacity. Additionally, it has also been argued that the account for the application of equivalent static force fraction in determining the capacity requires the involvement of demand calculation, which may be inappropriate. Therefore, the feasibility of the method needs to be further investigated.

Table 5- 2: Free body diagram of an exterior and interior joint bounded by the inflection points in columns and beams (M=0) (Tasligedik, A. S. and Pampanin, S) and formulation of joint capacity

Behaviour of structural elements under equivalent static forces	**Exterior Joint**	**Interior Joint**
	 $M_{cj}=\frac{\tau_{jt}}{\phi_1}+F_{jt}\cdot\phi_2,\ Units=\frac{kPa}{1/m^3}+kN\cdot m$ $\phi_1=\frac{2l_bl_c-l_ch_c-2l_bjd}{l_bjdb_ch_c(l_c-h_b)}$ $\phi_2=\frac{l_c-h_b}{2}\cdot\left[\frac{l_ch_c-2l_bl_c+4l_bjd}{4l_bl_c-2l_ch_c-4l_bjd}\right]$	 $M_{cj}=\frac{\tau_{jt}}{\phi_1}+F_{jt}\cdot\phi_2,\ Units=\frac{kPa}{1/m^3}+kN\cdot m$ $\phi_1=\frac{l_bl_c-l_ch_c-2l_bjd}{l_bjdb_ch_c(l_c-h_b)}$ $\phi_2=\frac{l_c-h_b}{2}\cdot\left[\frac{l_ch_c-l_bl_c+4l_bjd}{2l_bl_c-2l_ch_c-4l_bjd}\right]$

5.4. Evaluate Strength Hierarchy and Effect of Varying Axial Load (Local Level with Multiple Components, i.e. Subassembly Level)

In the current guidelines, the joint sway potential, i.e.$S_{Pij} = \frac{M_{bijl}+M_{bijr}}{M_{cijt}+M_{cijb}}$, is calculated for beam column joints in order to predict the location of hinge forming in the structure. As explained in Section 4.6, if the sway potential of one joint is found to be greater than 0.85, it then can be predicted that column hinges form at the top and/or the bottom of the joint region. However, this procedure only provides preliminary prediction, and more sophisticated procedures are required in order that the sequence of mechanisms at the joint region can be predicted.

As a result, the procedure to evaluate strength hierarchy and assess the sequence of mechanisms at beam-column joint region is proposed in this section. The mechanisms usually include beam flexural hinging, beam shear failure, column hinging, column shear failure, and joint shear failure, and the sequence these mechanisms is determined by the intersections of the capacity curves and the demand curves.

As shown in Figure 5- 8, three capacity curves are illustrated, representing beam flexural capacity, column flexural capacity and joint shear capacity. The yielding strength of the beam (i.e. half of the beam yielding strength if an exterior joint and full yielding strength if an interior joint) is constant independent of axial load. The procedures to determine column moment-axial load interaction curve and joint shear capacity (in terms of column moment versus axial load) are explained in Section 5.3.2 and 5.3.3. All the capacity curves are plotted in the same moment-axial load domain. It should be noticed that in some cases, where there is potentially beam shear failure or column shear failure, the capacity curves representing beam shear capacity and column shear capacity should also be plotted.

Also shown in Figure 5- 8, axial load demands N_g (i.e. G), with the vary ranges $\pm N_E$ (i.e. E), are plotted in the same moment-axial load domain. The gravity (plus live load) induced and earthquake induced axial load can be determined according to the latest load action design code, e.g. NZS1170.5. However, in the current guidelines, as discussed in Section 5.3.2, the earthquake induced axial load is estimated by the sum of exterior beam end shear capacities, however, it is worth noting that this load is "axial load demand at beam shear capacities", which is inappropriate to apply in strength hierarchy evaluation. In order to simplify the calculation process, the approach suggested by *Akguzel (2012)*, as shown in Figure 5- 7, can be applied. This approach assumes an inverted triangular profile of lateral load with an equivalent load F act at 2/3 of the total height. It is worth noting that as mentioned in Section 5.3.2, the impact of varying axial load can be assumed to be negligible for the interior joints in a multi-bay-frame. Thus, only N_g is considered for an interior joint, plotted as a single straight line in the moment-axial load domain.

With the capacity curves and the demand curves plotted in the same moment-axial load domain, the sequence of mechanisms then can be determined by the order of occurrence of intersections of the capacity and the demand. In Figure 5- 8, the strength hierarchy results for two exterior joints are illustrated. Since the lower level joint is subject to larger varying axial load demand compared to the upper level joint (i.e. the variation of axial load reduces with height), there is higher potentiality that the variation of axial load results in a change of sequence of mechanisms. As shown in Figure 5- 8, the first floor joint is expected to have mechanisms sequence changes from "beam hinging – joint shear failure – column flexural hinging" to "joint shear failure – column flexural hinging – beam flexural hinging" as the axial load demand decreases within the varying range. However, the upper floor joint is expected to have mechanisms sequence as "beam hinging – joint shear failure – column flexural hinging", and there is no alternation of sequence with the varying axial load demand.

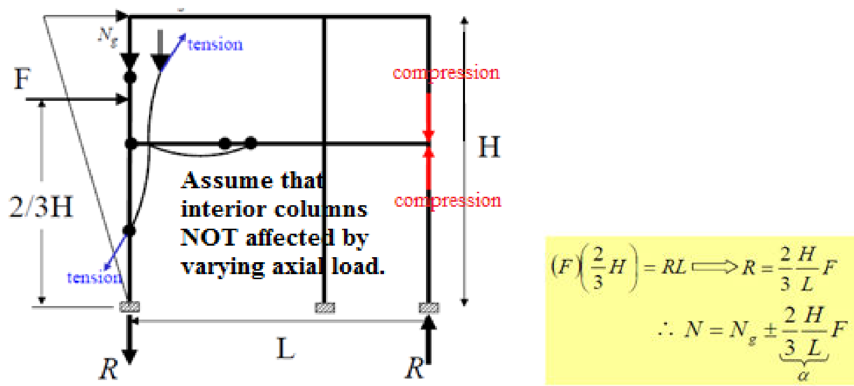


Figure 5- 7: Assumptions Made in Assessing Axial Load Variation (Akguzel, 2012)

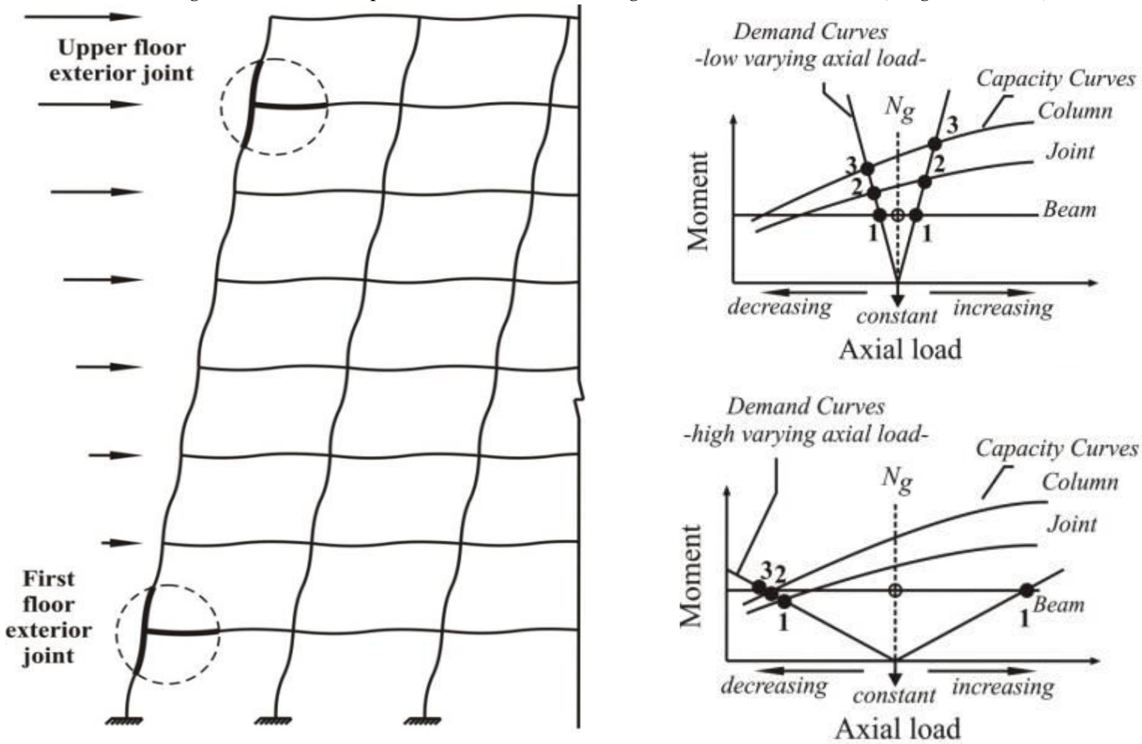


Figure 5- 8: Strength Hierarchy Theory (Courtesy of Umut Akguzel)

5.5. Global Structure Level

The current guidelines define three global structure mechanisms: beam sidesway mechanism, column sidesway mechanisms and mixed sidesway. In the guidelines, the evaluation of capacity of a frame structure corresponding to beam sidesway or column sidesway mechanism by hand is specified, i.e. SLaMa, the simple analytical pushover approach. However, the procedure to evaluate the capacity corresponding to mixed sidesway mechanism is not properly developed with details due to practical limitations associated with modelling and analysis, in spite of the fact that the mixed mode of structure response represents the most likely outcome. Hence, SLaMa approach (or the similarly simple but robust approaches) should be properly improved in order to provide good approximation of the real structure response without involving numerical modelling. The following improvements of the approach can be proposed, and it should be noticed that limitations of the current SLaMa are summarised in Section 4.6.3.

- To include the procedure to evaluate strength hierarchy as discussed Section 5.4
- To include the procedure to determine lower and upper bounds of lateral load capacity
- To include Portal Frame Method if applicable
- To modify the formulae applied during calculation, together with the suggested values for some coefficients or parameters

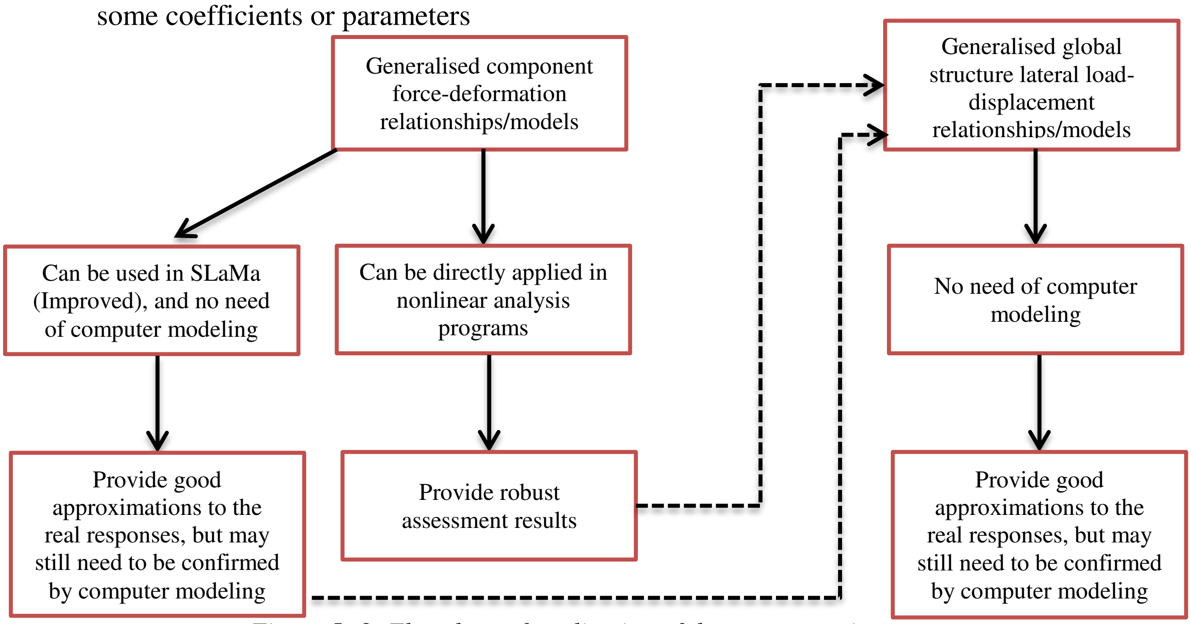


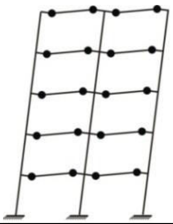
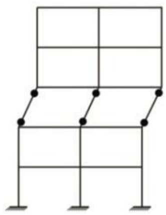
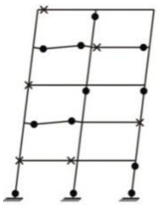
Figure 5- 9: Flowchart of application of the representative curves

Optimally, generalised global structure lateral load – displacement models (e.g. summarised and averaged for common building typologies based on a great number of building case studies, experimental work and modelling work), together with generalised component force-deformation models (e.g. summarised and averaged for typical component types based on a great number of component case studies, experimental work and modelling work) should be developed. The flowchart (Figure 5- 9) showing the application of these models is presented. More explanations regarding the models and about the process in Figure 5- 9 are shown in Section 5.5.3 and 5.6. However, due to research timeframe constraint, the task to establish these relationships has not been accomplished.

5.5.1. Determination of Lower and Upper Bounds of Lateral Load Capacity

A lower bound and an upper bound of the lateral load capacity of a frame can be established, providing a varying range of the capacity if the frame is expected to have a mixed sidesway mechanism. The upper bound and the lower bound can be determined corresponding to a beam sidesway mechanism and a column sidesway mechanism, as shown in Table 5- 3and Figure 5- 10.

Table 5- 3: Calculation of upper and lower bound of lateral load capacity

Upper bound	**Lower bound**	**In Between**
Beam Sidesway Mechanism 	Column Sidesway Mechanism 	Mixed Sidesway Mechanism 
$\mathrm{OTM} = \sum_i \mathrm{M_{coli}} + \left(\sum_n \mathrm{V_{end\ beam,n}}\right) \mathrm{L}$ $\mathrm{V_b} = \frac{\mathrm{OTM}}{\mathrm{h_{eff,beam\ sidesway}}}$	$\mathrm{OTM} = \sum_i \mathrm{M_{coli}}$ $\mathrm{V_b} = \frac{\mathrm{OTM}}{\mathrm{h_{eff,col\ sidesway}}}$	$\mathrm{OTM} = \sum_i \mathrm{M_{coli}} + \left(\sum_x \mathrm{V_{end\ beam,x}}\right) \mathrm{L}$ $\mathrm{V_b} = \frac{\mathrm{OTM}}{\mathrm{h_{eff,mixed\ sidesway}}}$

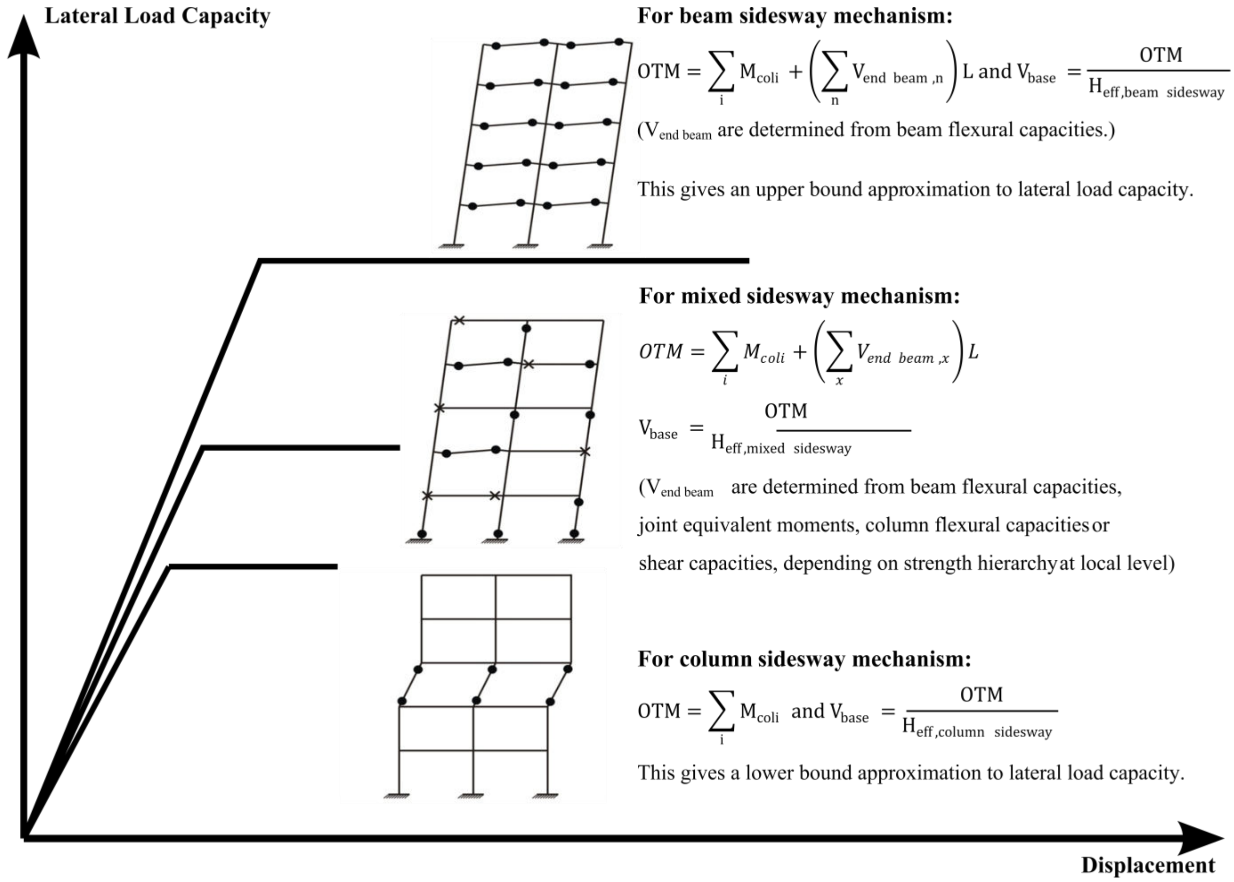


Figure 5- 10: Lateral load capacity versus displacement for different global mechanisms

where $\sum_i M_{coli}$ = Sum of base column moments

$\sum_n V_{end\ beam,n}$ = Sum of end beam shears for all n levels

L = Frame full span

Determination of lateral load capacity:

From the detailed procedure of the current SLaMa shown in Section 4.6, it has been found that there is no specification associated with the reduction of earthquake induced axial load contribution (i.e. N_EL) to the total overturning moment especially under column sidesway mechanism circumstances. Hence, if the current procedure is performed, it may end up with similar base shear capacities independent of mechanisms, which is obviously incorrect. Hence, the following modification should be proposed:

- For beam sidesway mechanism, all exterior beam shear capacities contribute to the total overturning moment.
- For column sidesway mechanism at the first level, no earthquake induced axial load contribution should be considered.
- For mixed sidesway mechanism, $V_{end\ beam}$ should be determined from beam flexural capacities, or joint equivalent moments, or column flexural capacities, or column shear capacities, depending on strength hierarchy at local level, as illustrated in Figure 5- 11.

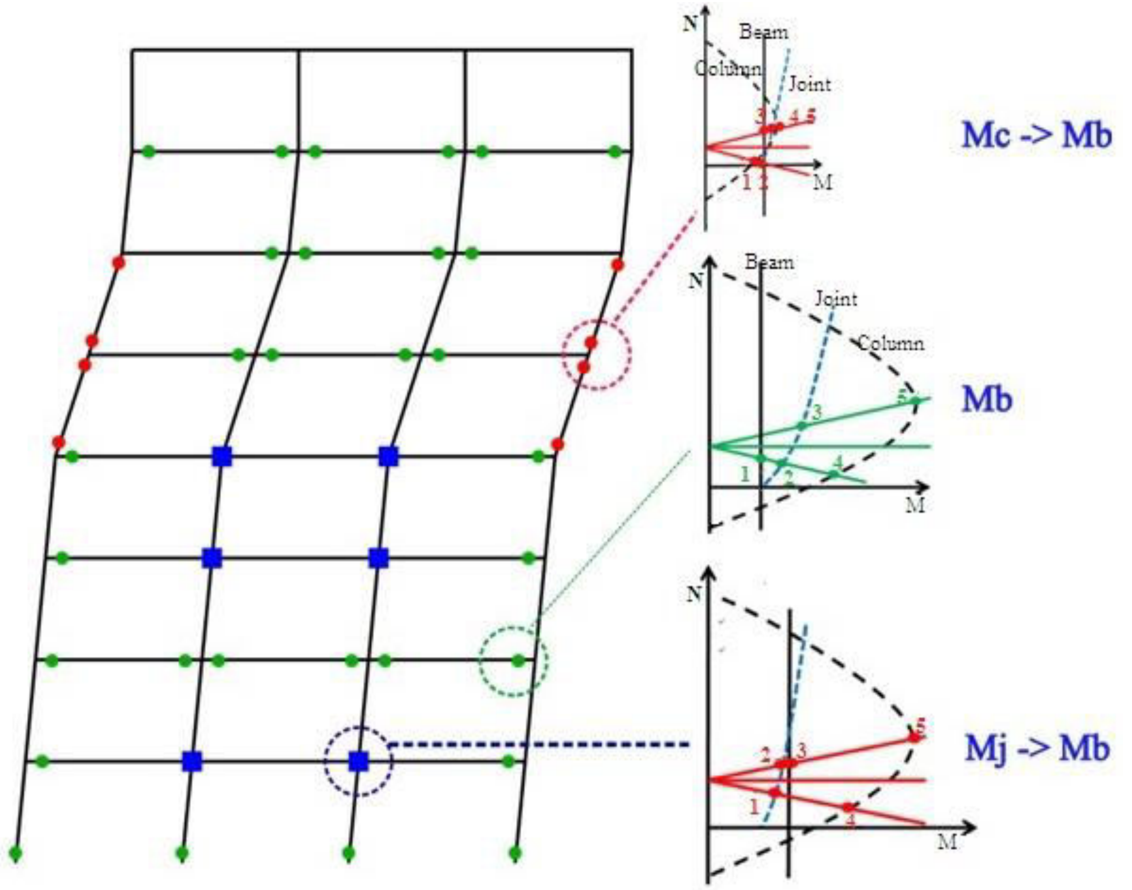


Figure 5- 11: Determination of global mechanism based on evaluation of strength hierarchy at local level

It is worth noting that strength degradation should be considered if shear failures of the assessed components are anticipated (i.e. the checks for shear are not satisfying). The procedure to perform the shear check is shown in Section 4.6; however, such procedure may be subject to modification and improvement according to the latest researches (e.g. target to provide component force-deformation relationship together with flexural-shear interaction).

The determination of effective height, yield displacement, displaced shape, ductility capacity is discussed in the following paragraphs.

Determination of effective height:

The followings are defined in the current guidelines to determine effective height of the structure:

$$h_{eff} = \frac{\sum m_j h_j^2}{\sum m_j h_j} \text{ (Force − based approach)}$$

$$h_{eff} = \frac{\sum m_i \Delta_i h_i}{\sum m_i \Delta_i} \text{ (Displacement − based approach)}$$

However, in NZSEE 2006 Section 7.2.4, the procedure shown in Table 5- 4 is specified, which has been found inconsistent with the procedure in SLaMa.

Table 5- 4: Determination of effective height from NZSEE 2006 Section 7.2.4

Beam Sidesway Mechanism	Column Sidesway Mechanism
$h_{eff} = 0.64H, \text{for } n \leq 4$ $h_{eff} = [0.64 - 0.0125(n-4)]H, \text{for } 4 < n \leq 20$ $h_{eff} = 0.44H, \text{for } n \geq 4$ (consistent with the procedure adopted in Displacement Based Design)	$H_{eff} \approx 0.5H$, or more accurately, $h_{eff} = \left(0.64 - 0.14\frac{\mu_s - 1}{\mu_s}\right)H,$ and $\mu_s = 1 + \frac{(\varphi_u - \varphi_y)L_p H}{n\Delta_y}$

It has been found that the variation of effective height has significant impact on the determination of lateral load capacity. Therefore, clarification is required for the procedures to determine effective height.

Determination of yield displacement:

The following formula is specified in the current guidelines to determine yield displacement:

$$\Delta_y = 0.5\varepsilon_y \frac{L_b}{h_b} h_{eff} \text{ or } [0.5\varepsilon_y \frac{L_b}{h_b} h_{eff}] \frac{OTM_1}{OTM_2}$$

However, in "*Seismic Design of Reinforced Concrete and Masonry Buildings" (Paulay, T. and Priestley, M.J.N.)*, the determination of yield displacement is specified as following and is shown in Table 5- 5, which is inconsistent with the procedure specified in NZSEE 2006. Therefore, clarification is required for the procedures to determine yield displacement.

$$\Delta_y = \frac{l_c^2}{6}\sum_1^n \varphi_{yci}$$

(where it is assumed that the points of contra-flexure occur at mid column height for all stories)

Table 5- 5: Determination of yield displacement from "Seismic Design of Reinforced Concrete and Masonry Buildings" for different global structure mechanisms

Beam Sidesway Mechanism	Column Sidesway Mechanism
$\theta_{yb} = \frac{1}{2}\varphi_{yb}\frac{L_b}{2}$ $\theta_{yb-c} = \theta_{yb}$ $\Delta_y = \theta_{yb-c}(\frac{2}{3}h_{eff}) = \left(\frac{1}{2}\varphi_{yb}\frac{L_b}{2}\right)(\frac{2}{3}h_{eff})$ Assumptions: • The contra-flexural point occurs at beam mid span. • Same rotation along the column direction and along the beam direction for the yield state • φ_{yb} should be chosen as the most critical beam yielding curvature	$\theta_{yc} = \frac{1}{2}\varphi_{yc}\frac{L_c}{2}$ $\Delta_y = \theta_{yc}(\frac{2}{3}h_{eff}) = (\frac{1}{2}\varphi_{yc}\frac{L_c}{2})(\frac{2}{3}h_{eff})$ Assumptions: • The contra-flexural point occurs at column mid height. • φ_{yc} should be chosen as the column yielding curvature at the level where sidesway mechanism occurs.

<u>Determination of displaced shape</u> (to be applied in displacement-based approach):

The displacement-based assessment approach requires an estimation of displaced shape of a structure. However, the current guidelines lack detailed specification regarding this issue. In "*Displacement-Based Seismic Design of Structure" (Priestley, M.J.N., Calvi, G.M., Kowalsky, M.J.)*, the procedure to approximate the displaced shape of a frame structure is shown as following. However, such procedure is only applicable if the frame is subject to a beam sidesway mechanism.

$$\Delta_i = \frac{4}{3}\left(\frac{H_i}{H_n}\right)\left(1 - \frac{H_i}{4H_n}\right) \text{ for } n > 4$$

$$\delta_i = \frac{H_i}{H_n} \text{ for } n < 4$$

$$\Delta_i = \delta_i\left(\frac{\Delta_c}{\delta_c}\right)$$

In "*Seismic Design of Reinforced Concrete and Masonry Buildings*" (*Paulay, T. and Priestley, M.J.N.*), the derivation of displacements of all levels, for frame structures subject to either beam sidesway mechanisms or column sidesway mechanisms, is found, and the detailed procedure is shown in Table 5- 6. However, no procedure was found to calculate the displaced-shape of a frame with a mixed sidesway mechanism.

Table 5- 6: Determination of displaced shape of the structure from "Seismic Design of Reinforced Concrete and Masonry Buildings" for different global structure mechanisms

Beam Sidesway Mechanism	Column Sidesway Mechanism
The displacements for all levels can be directly calculated without adopting the concept of displaced-shape ratio and critical level. $\Delta_i = \Delta_{yi} + \Delta_{pi} = \theta_{yci}\left(\frac{2}{3}L_c\right) + \theta_{pci}(L_c - 0.5L_{pc})$ $\theta_{yci} = \theta_{ybi} = \frac{1}{2}\varphi_{ybi}\frac{L_b}{2}$ $\theta_{pbi} = (\varphi_{ubi} - \varphi_{ybi})L_{pb}$ and $L_{pb} = 0.08L + 0.022f_y d_b$ $\theta_{pci} = \frac{L'}{L_b}\theta_{pbi}$ If the calculation of displaced-shape ratio is	Assume column sway occurs at level i: From Level 1 to Level (i-1): $\theta_{yci} = \frac{1}{2}\varphi_{yci}\frac{L_c}{2}$ $\Delta_1 = \Delta_{yc1} = \theta_{yc1}\left(\frac{2}{3}L_c\right) = \frac{1}{2}\varphi_{yc1}\frac{L_c}{2}\frac{2}{3}L_c = \frac{L_c^2}{6}\varphi_{yc1}$ $\Delta_2 = \Delta_{yc2} = \frac{L_c^2}{6}(\varphi_{yc1} + \varphi_{yc2})$ $\Delta_{(i-1)} = \Delta_{yc(i-1)} = \frac{L_c^2}{6}\sum_{1}^{i-1}\varphi_{yc}$ From Level i to Level n:

required: (assume level 1 to be the critical level) $\Delta_c = \Delta_{yc} + \Delta_{pc} = \theta_{ycc}\left(\frac{2}{3}L_c\right) + \theta_{pcc}(L_c - L_{pc})$ $\theta_{ycc} = \theta_{ybc} = \frac{1}{2}\varphi_{ybc}\frac{L_b}{2}$ $\theta_{pbc} = (\varphi_{ubc} - \varphi_{ybc})L_{pb}$ and $L_{pb} = 0.08L + 0.022f_y d_b$ $\theta_{pcc} = \frac{L'}{L_b}\theta_{pbc}$ Displaced shape ratio at level i $= \frac{\Delta_i}{\Delta_c}$	$\theta_{pci} = (\varphi_{cui} - \varphi_{cyi})L_{pci}$ $L_{pci} = 0.08L + 0.022f_y d_{ci}$ $\Delta_{pi} = \theta_{pci}(L_c - L_{pci}) = (\varphi_{cui} - \varphi_{cyi})L_{pci}(L_c - L_{pci})$ $\Delta_i = \Delta_{yci} + \Delta_{pi} = \left(\frac{L_c^2}{6}\sum_1^i \varphi_{yc}\right) + (\varphi_{cui} - \varphi_{cyi})L_{pci}(L_c - L_{pci})$ $\Delta_{(i+1)} = \Delta_{yc(i+1)} + \Delta_{pi} = \left(\frac{L_c^2}{6}\sum_1^{i+1} \varphi_{yc}\right) + (\varphi_{cui} - \varphi_{cyi})L_{pci}(L_c - L_{pci})$ $\Delta_n = \Delta_{ycn} + \Delta_{pi} = \left(\frac{L_c^2}{6}\sum_1^{n} \varphi_{yc}\right) + (\varphi_{cui} - \varphi_{cyi})L_{pci}(L_c - L_{pci})$

L_b = Beam full length
L = Half of clear beam span and assuming that the contra-flexure point locates at the mid span
L' = Distance between the two beam hinges, which is assumed to be 2/3 of beam full length, i.e. $L'/L_b=2/3$
δ_i = Displaced shape ratio at level i
δ_i = Displaced shape ratio at critical level
H_i = Height of level i
H_n = Height of the structure
Δ_i = Displacement at level i
Δ_c = Displacement at critical level (assumed to be level 1)
Δ_{yc} = Yield displacement at critical level
Δ_{pc} = Plastic displacement at critical level
L_c = Column full height
L_{pb} = Beam plastic hinge length
L_{pc} = Column plastic hinge length (assumed to be 0 for beam sidesway mechanism)
θ_{ycc} = Yield rotation at critical level along column direction
θ_{pcc} = Plastic rotation at critical level along column direction
θ_{ybc} = Yield rotation at critical level along beam direction
θ_{pbc} = Plastic rotation at critical level along beam direction
φ_{ybc} = Beam yield curvature at critical level
φ_{ubc} = Beam ultimate curvature at critical level

Determination of ductility capacity:

In the current guidelines, the displacement ductility capacity is determined based on the assessed global mechanism, shown in Figure 4- 6 and Table 4- 21 in Section 4.6.2.8. However, the following formulae are usually adopted in displacement-based approach, found in "*Displacement-Based Seismic Design of Structure" (Priestley, M.J.N., Calvi, G.M., Kowalsky, M.J.).*

$$\Delta_{UC} = \frac{\sum m_i \Delta_i^2}{\sum m_i \Delta_i}$$

$$\mu_{sc} = \frac{\Delta_{UC}}{\Delta_y}$$

This alternative method may be valid if the displaced shape of the structure is well approximated.

5.5.2. Computation of Pushover Curve with Sequence of Mechanisms by Portal Frame Method

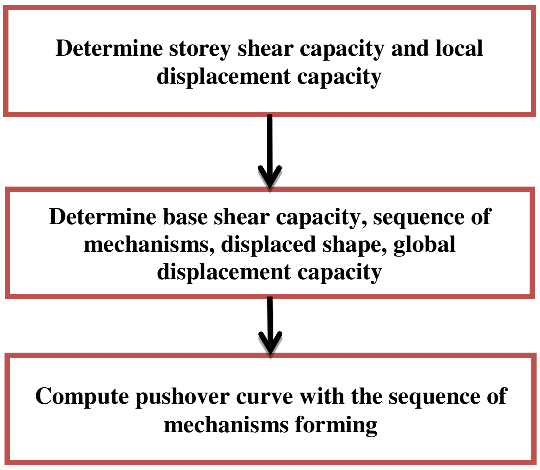


Figure 5- 12: Flowchart showing a summarised procedure of Portal Frame Method

As shown in Figure 5- 12, the procedure of Portal Frame Method can be summarised as "three-steps". The first step involves the determination of storey shear capacity and storey displacement capacity based on the capacity calculation of individual components and strength hierarchy evaluation, followed by the second step, in which base shear capacity of the global structure, displaced shape (and global displacement capacity) and the sequence of mechanisms are derived. At the last step, a pushover curved can be computed with the development of mechanisms. This method does not involve numerical modelling.

5.5.2.1. Determination of Storey Shear Capacity and Local Displacement Capacity

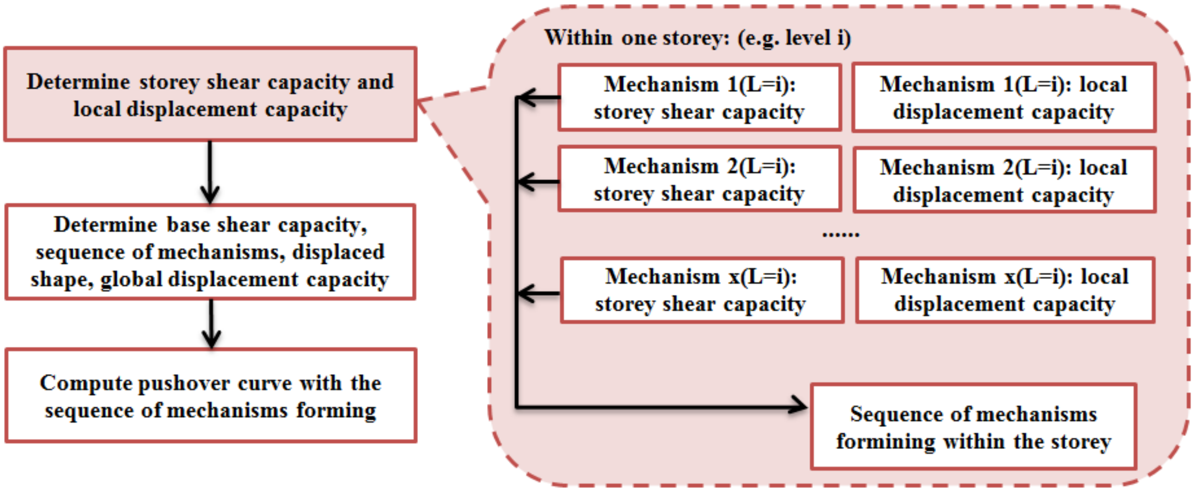


Figure 5- 13: Detailed procedure to determine storey shear capacity and local displacement capacity

At local storey level, the hierarchy of strength of components are assessed, and the sequence of mechanisms then can be determined. It is worth noting that the determined sequence should be cross-checked by the evaluation of hierarchy of strength within each beam-column joint (i.e. at subassembly level) following the procedure presented in Section 5.4.

Within level i, local mechanisms include:

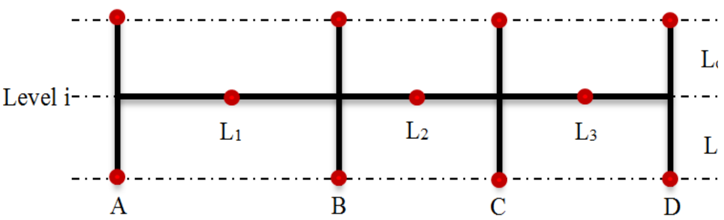
- Beam flexural hinging (i.e. beam reaching flexural capacity which is estimated following the improved guidelines, see Section 4.6.2 and 5.3.1);
- Column flexural hinging (i.e. column reaching flexural capacity which is estimated following the improved guidelines, see Section 4.6.2 and 5.3.2);
- Joint shear failure (i.e. joint reaching shear capacity which is estimated following the procedure shown in Section 5.3.3);
- Column shear failure (i.e. column reaching shear capacity which is estimated following the improved guidelines, see Section 4.6.2 and 5.3.2)
- Beam shear failure (i.e. beam reaching beam shear capacity which is estimated following the improved guidelines, see Section 4.6.2 and 5.3.1)

It is worth noting that the beam shear failure mechanism is not usually accounted for as it is expected to be the least likely failure mechanism compared to the others, except for some special cases where beam shear failure is anticipated (i.e. beams may not have sufficient shear capacity).

The mechanisms listed above can be categorised into two main types: the flexural type of mechanism and the shear type of mechanisms. The procedures to determine storey shear and local displacement for different types of mechanisms are shown in the following.

<u>Flexural type of mechanisms:</u>

The local storey shears corresponding to the flexural type of mechanisms can be determined following the procedure shown in the following.



$L_{1,2,3}$ = Beam 1,2,3 full length
L_c = Column height
L_T = Frame full span
$V_{a,b,c,d}$ = Shear at column A,B,C,D
$M_{a,b,c,d}$ = Moment at column A,B,C,D
V_i = Sum of storey shears of level i and above

$$V_a = \frac{V_i}{L_T}\frac{L_1}{2}, M_a = V_a\frac{L_c}{2}, \text{hence}, M_a = \frac{V_i}{L_T}\frac{L_1}{2}\frac{L_c}{2}$$

$$V_b = \frac{V_i}{L_T}\frac{L_1 + L_2}{2}, M_b = V_b\frac{L_c}{2}, \text{hence}, M_b = \frac{V_i}{L_T}\frac{L_1 + L_2}{2}\frac{L_c}{2}$$

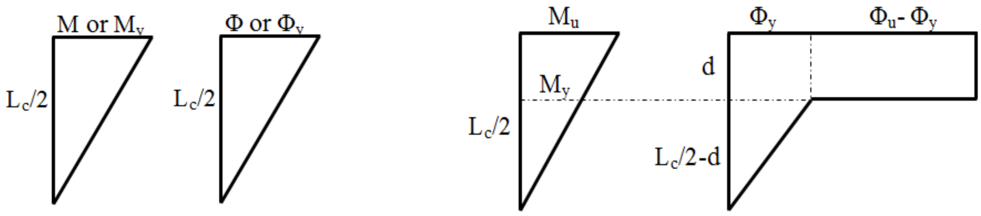
$$V_c = \frac{V_i}{L_T}\frac{L_2 + L_3}{2}, M_c = V_c\frac{L_c}{2}, \text{hence}, M_c = \frac{V_i}{L_T}\frac{L_2 + L_3}{2}\frac{L_c}{2}$$

$$V_d = \frac{V_i}{L_T}\frac{L_3}{2}, M_d = V_d\frac{L_c}{2}, \text{hence}, M_d = \frac{V_i}{L_T}\frac{L_3}{2}\frac{L_c}{2}$$

As shown as "red dots" in the illustration above, it is assumed that contra-flexure points occur at mid beam or column spans. It is also assumed that the storey shear is distributed according to the tributary area (length) assumption. Moments at column A, B, C, D are determined based on the calculated beam and column flexural capacities. With the determined M_a, M_b, M_c and M_d applied in the equation listed above, the sum of storey shears of level i and above, V_i, can be determined, corresponding to each flexural mechanism at level i (i.e. beam BC hinging, beam AB or CD hinging, column B or C hinging, column A or D hinging). By listing all calculated V_i values in "the-smallest-to-the-largest" order, the sequence of the flexural mechanisms can also be determined.

As discussed in Section 5.3.1 and 5.3.2, for simplification purpose, it can be assumed that beam and column sections have bilinear moment-curvature relationships without consideration of strength degradation and residual strength. It can also be assumed that the earthquake induced axial loads are resisted only by exterior columns; hence, moment-curvature relationships under $G+\Psi_cQ\pm E$ are applied for exterior columns, and moment-curvature relationships under $G+\Psi_cQ$ for interior columns. With the applied section moment-curvature relationships (including information of yield and ultimate curvature), local deformation can be estimated by the procedure shown as follows.

Case 1: columns pre-yield or just yield   Case 2: columns post-yield



$$\frac{\Delta}{2} = \left(\frac{1}{2}\Phi\frac{L_c}{2}\right)\left(\frac{2}{3}\frac{L_c}{2}\right) = \frac{1}{6}\Phi\left(\frac{L_c}{2}\right)^2$$

$$\frac{\Delta}{2} = \left(\frac{1}{2}\Phi_y\left(\frac{L_c}{2} - d\right)\right)\left(\frac{2}{3}\left(\frac{L_c}{2} - d\right)\right) + (\Phi_u d)\left(\left(\frac{L_c}{2} - d\right) + \frac{d}{2}\right)$$

**φ is determined by linear extrapolation of moment-curvature relationship of the member.*
**d is determined by linear extrapolation of moment profile of the section.*

This procedure, based on Moment Area Theorem, is simple and straightforward. However, the assumptions and simplification applied in the procedure may result in not as good estimation of local displacement as expected. Some of the assumptions and limitations are listed:

- It is assumed that the assessed level i undergoes the same displacement calculated at column, without considering diaphragm flexibility or deformability.
- It is assumed that the greater flexural rigidity of members that have not cracked and the greater flexural rigidity between cracks can be ignored. Therefore, the elastic deformation due to flexure will be somewhat over-estimated *(Paulay, T. and Priestley, M.J.N.)*.

Referring to "*Seismic Design of Reinforced Concrete and Masonry Buildings" (Paulay, T. and Priestley, M.J.N),* it is assumed that when the seismic loading on the frame is increased until yielding occurs, and yielding will commence at all the critical sections at the same load and at sufficient sections to form mechanism. However, this condition rarely occurs in practice because of:

- Variations in the actual strengths of materials;
- Differences between the approximate triangular code-specified seismic loading and the actual distribution of inertia loading induced in the structure by an earthquake;
- Various factors affecting the strength of members

The Portal Frame Method at this step enables simple estimation of local storey shear and displacement. Future researches and investigations are required in aiming to provide more accurate solutions.

Shear type of mechanisms:

For joint shear failure mechanism, the storey shear can be determined from the sum of equivalent moments of all joints at the assessed level, $V_{storey} = \sum \frac{M_{joint}}{\frac{L_c}{2}}$, and the joint equivalent moments are determined following the procedure shown in Section 5.3.3.

As for column shear failure mechanism, the storey shear can be determined from the sum of shear capacities of all columns at the assessed level, $V_{storey} = \sum V_{CPI}$, and the column capacities are determined following the procedures discussed in Section 4.6.2 and Section 5.3.2.

The estimation of shear deformation is not included, since it may require very complicated calculation, and can be very time-consuming and inaccurate if only hand calculation is applied. The neglect of displacement due to shear can be to some extent compensated by the overestimate of elastic flexural displacements, referring to "*Seismic Design of Reinforced Concrete and Masonry Buildings" (Paulay, T. and Priestley, M.J.N.).*

5.5.2.2. Calculate Base Shear Capacity and Global Displacement Capacity

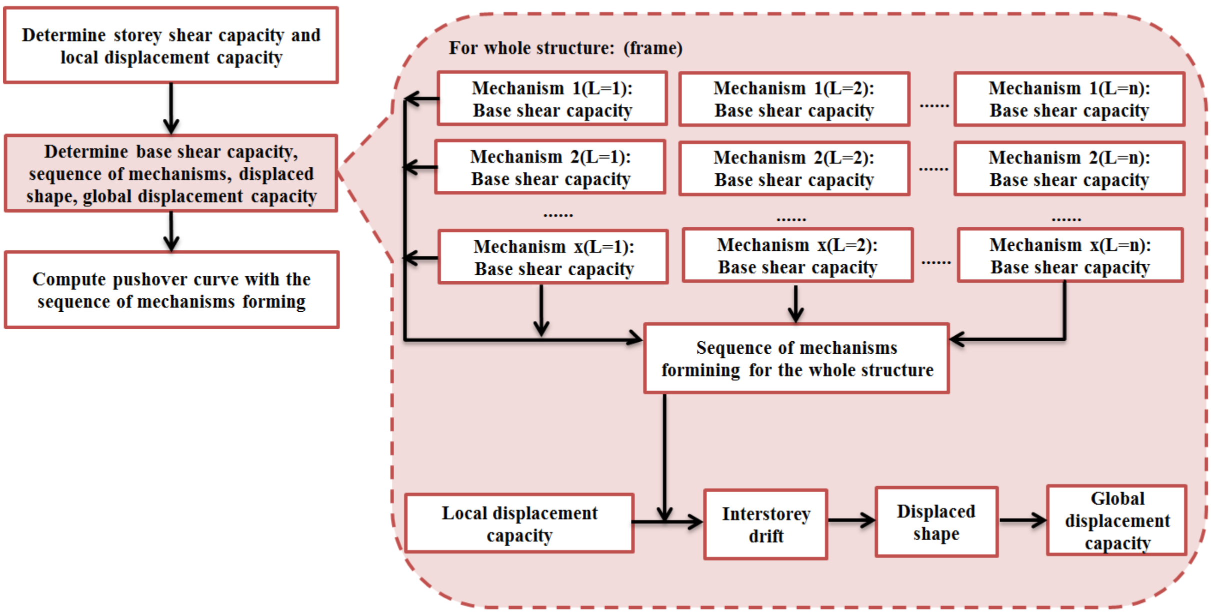
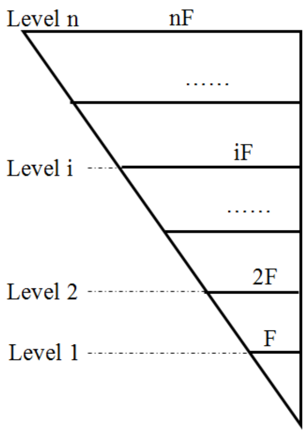


Figure 5- 14: Detailed procedure to determine base shear capacity, sequence of mechanisms, and global displacement capacity

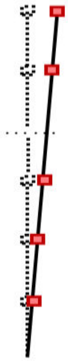


$$V_{\text{base shear}} = \frac{[1 + 2 + 3 + \cdots + n]F}{[i + (i+1) + (i+2) + \cdots + n]F} V_i = \frac{\sum_{n=1}^{n} n}{\sum_{n=i}^{n} n} V_i$$

Figure 5- 15: Assumed static earthquake force profile

In order to calculate base shear corresponding each mechanism, for simplification purposes, an inverted triangular profile of static earthquake forces can be assumed, as shown in Figure 5- 15, and the base shear capacity can be calculated by applying the formula presented. It is worth noting that the assumption of the inverted triangular profile is not appropriate when post-elastic mechanisms have reached. In reality, the external force profile varies with the change of the displaced shape of the structure. The impact of this assumption needs to be assessed by comparing results from Portal Frame Method and from numerical adaptive pushover analysis. With the base shear capacities determined corresponding to all the mechanisms, the sequence of mechanisms forming can thereby be determined by sorting the base shear capacities from the smallest to the largest.

Before computing the displacement corresponding to each mechanism at the top of a frame structure, the displaced shape needs to be determined first, based on the calculation of local storey displacement. The displaced shape at first yield state (i.e. yielding mechanism "level x beams reaching yielding flexural capacity") is assumed to be in a linear profile, as shown in Figure 5- 16. In other words, the interstorey drifts are the same for all levels, being equal to the calculated yielding rotation at level x. The calculation process to estimate the displacements at all level based on interstorey drifts is presented below the figure.



$$\theta_i = \theta_{y,x}$$

$$\delta_i = \frac{\sum_1^i \theta_n}{\sum_1^n \theta_n}$$

$$\Delta_i = \delta_i \frac{\Delta_1}{\delta_1}$$

$$\Delta_n = \delta_n \frac{\Delta_1}{\delta_1}$$

θ_i = Interstorey drift at level i
$\theta_{y,x}$ = Yield rotation calculated at level x
δ_i = Displaced shape ratio at level i
$\sum_1^n \theta_n$ = Sum of interstorey drifts of all levels, i.e. from level 1 to level n
$\sum_1^i \theta_n$ = Sum of interstorey drifts from level 1 to level i
Δ_i = (Global) displacement at level i
Δ_1 = (Global) displacement at level 1
Δ_n = (Global) displacement at top (level n)
δ_1 = Displaced shape ratio at level 1
δ_i = Displaced shape ratio at level i
δ_n = Displaced shape ratio at top (level n)

Figure 5- 16: Linear displaced shape profile at first yield state

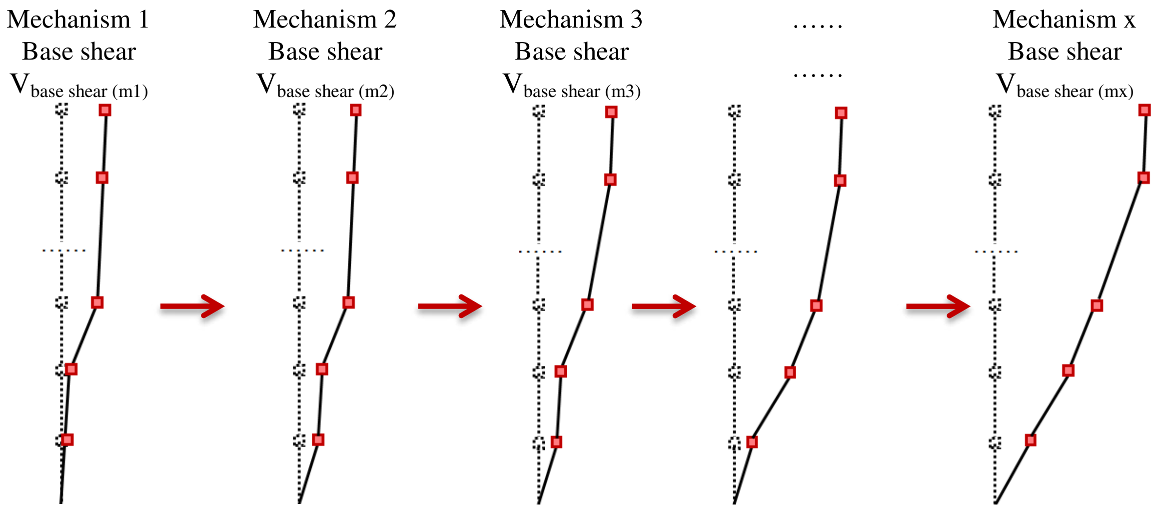


Figure 5- 17: Development of displaced shape corresponding to the sequence of mechanisms

$$\theta_i = \frac{\Delta_{local,i}}{L_c}$$

θ_i	=	Interstorey drift at level i
$\Delta_{local,i}$	=	Local displacement at level i
L_c	=	Storey height

For the post-yield states after the "first yield" state, following the determined mechanisms sequence as shown in Figure 5- 17, the increase of interstorey drift at level i can be calculated from the local (storey) displacement which corresponds to each of the mechanisms. The derivation of the displaced shape and displacement at each level follows the same procedure for the "first yield" state.

The determination of the failure mechanism (Mechanism x in Figure 5- 17) is vital. It is acknowledged that one of the most critical issues of the application of Portal Frame Method is to correctly determine the failure mechanism. Otherwise, the capacity can be either over-estimated or under-estimated. It should be assured that no further mechanisms with larger lateral load (base shear) capacity can develop, and that the mechanisms developed before Mechanism x will not trigger failure of the structure until Mechanism x. The failure mechanism should be determined corresponding to the limit states defined at global level (e.g. interstorey drift). In the current NZSEE 2006 Guidelines, only Ultimate Limit State is considered, under which an interstorey drift of 2.5% is defined. Alternatively, as shown in "*Modelling of Shear Hinge Mechanisms in Poorly Detailed RC Beam-Column Joints" (Pampanin, S., Megenes, G., Carr, Athol.)*, the interstorey drift limits corresponding to different limit states are specified in Table 5- 7.

Table 5- 7: Specification of interstorey drifts corresponding to different limit states

Limit State	Drift (%)
First diagonal cracking	0.65
Extensive damage	1.0
Critical damage (reparability issues)	1.5
Incipient collapse	2

However, different from NZSEE Guidelines where the limit state criteria are specified at global level, in ASCE 41-13, the limit state and acceptance criteria are defined on component basis, shown in

Table in Section 3.3.4. Future researches and investigations are required regarding this issue, and limit states at material level, component level, or global level should be properly defined.

5.5.2.3. Compute Pushover Curve with Sequence of Mechanisms

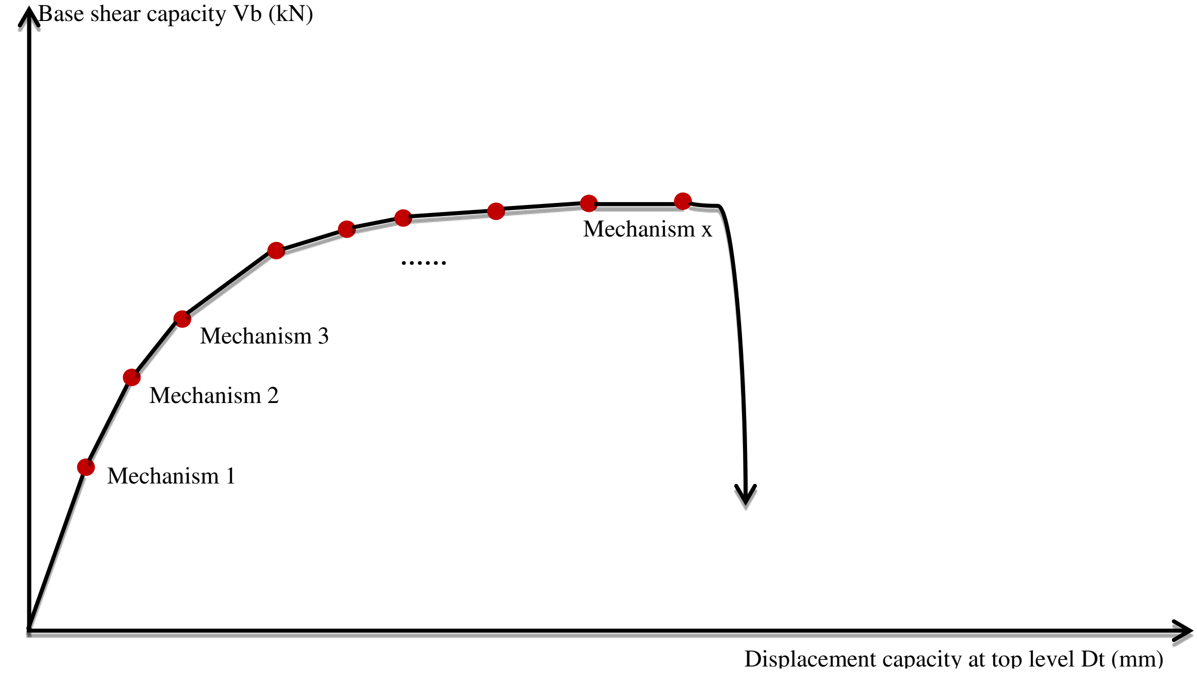


Figure 5- 18: Illustration of computing pushover curve with sequence of mechanisms shown

As shown in Figure 5- 18, the lateral load capacity and the displacement at the top of the structure corresponding to all assessed mechanism are plotted. The capacities (both lateral load and displacement) corresponding to Mechanism x are defined as the ultimate capacities, and Figure 5- 19 illustrates several ways to approximate yielding displacement and lateral load from a pushover plot.

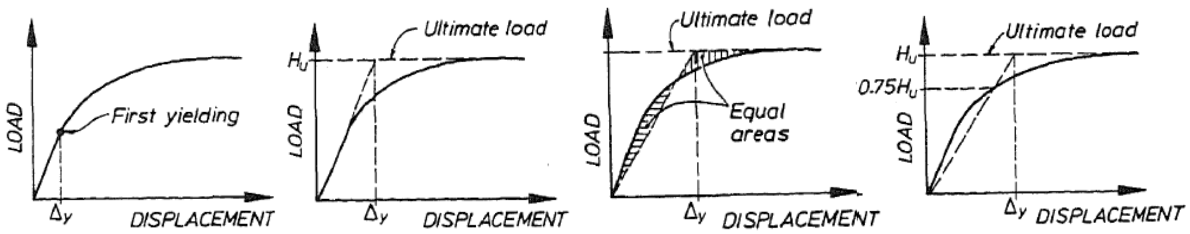


Figure 5- 19: Estimation of yielding displacement based on (a) steel reinforcement "first yield"; (b) equivalent elasto-plastic yield; (c) equivalent elasto-plastic energy absorption; (d) reduced stiffness equivalent elasto-plastic yield (Park, R., 1988)

The current guidelines adopt the "equivalent elasto-plastic energy absorption" (i.e. "equivalent area") procedure to approximate yielding capacities.

5.5.2.4. Global yield State Assumption

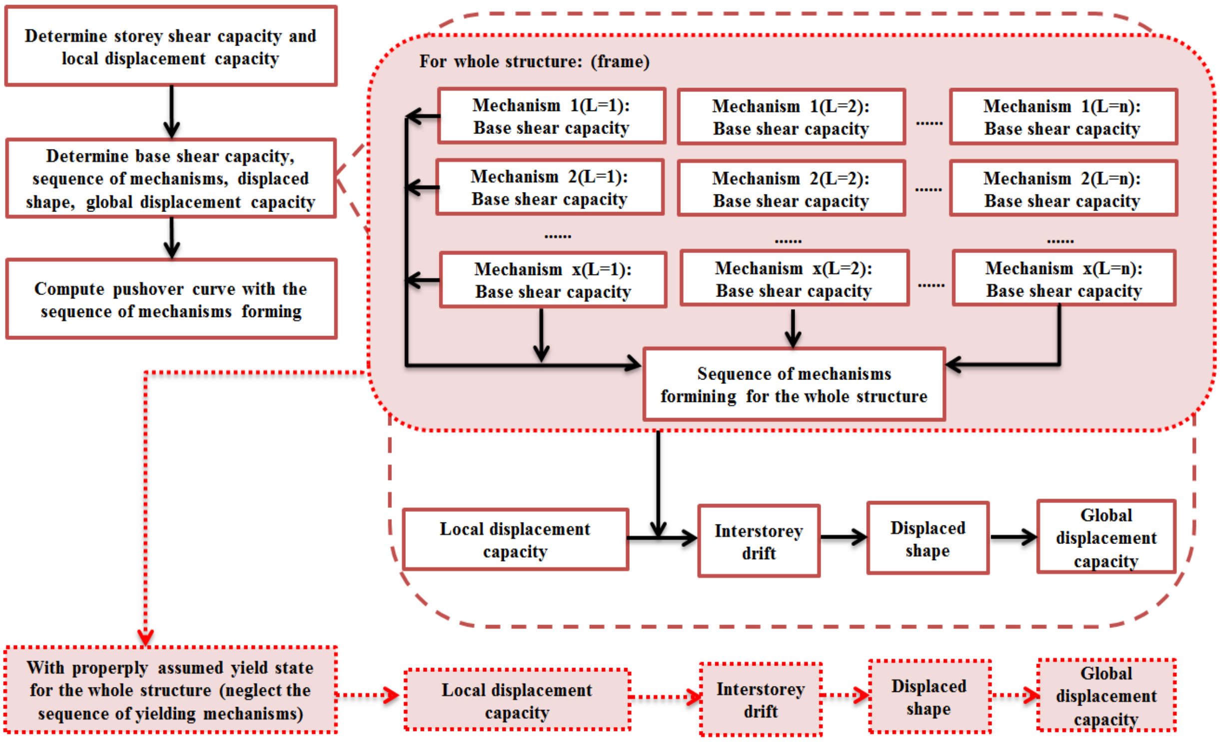


Figure 5- 20: Simplification of assuming a general yield state

Compared to the mechanisms concerning shear-type of failures, the sequence of the formation of initial flexural hinges is not important. For common cases, the mechanisms such as beam flexural hinging or column flexural hinging occur at the beginning after the “first-yield” state, and these flexural-type of mechanisms can be replaced by one “general yield state” ignoring the order of these mechanisms. Such simplification may lead to differences in the estimation of capacities, and the differences are shown in Chapter 7. This simplified procedure may be necessary if a large number of building cases are to be assessed.

5.5.2.5. Displaced Shape (Determined from Portal Frame Method) for Mixed Sidesway Mechanism

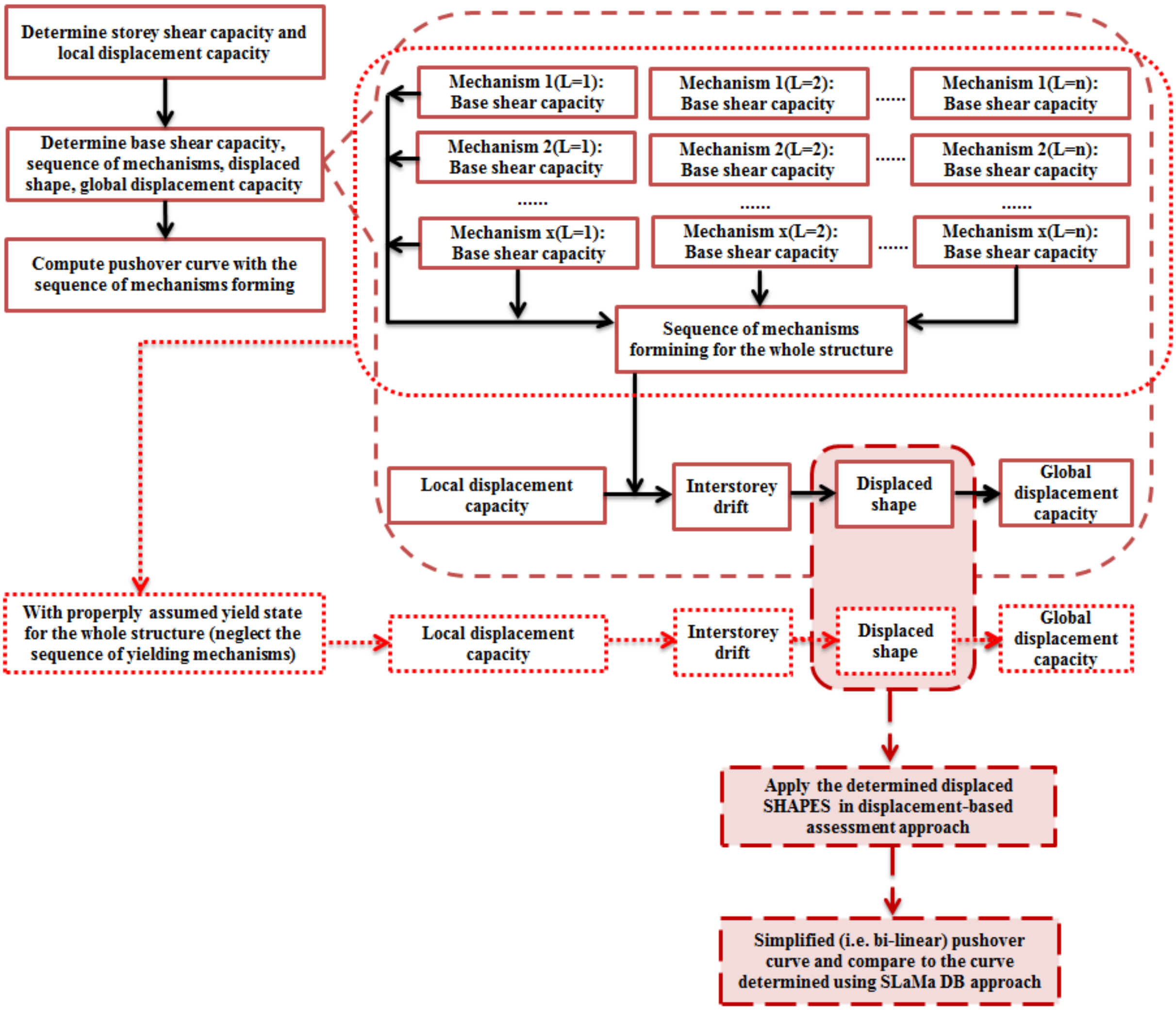


Figure 5- 21: Application of displaced shape for mixed sidesway mechanism in the displacement-based assessment procedure

As presented in Section 5.5.2.2, Figure 5- 17 shows the development of displaced shapes following the sequence of mechanisms. It is expected that the displaced shape corresponding to the failure mechanism (and other critical mechanisms that may trigger partial failure of the structure), which is highly likely to be a mixed sidesway mechanism, can be adopted in the displacement-based assessment approach. With the chosen displaced shape applied in the DB approach, a simplified bi-linear pushover plot can then be obtained. This simplification may be necessary when dealing with a large number of building cases. The accuracy of the procedure depends on the choice of the displaced shapes (i.e. determination of failure mechanism), which is discussed in Section 5.5.2.2.

The computed bi-linear pushover plot can then be compared to the lower and upper capacity curves determined following the procedure in Section 5.5.1, for the purpose of confirming that the computed pushover curve is indeed bounded by the upper and lower bounds.

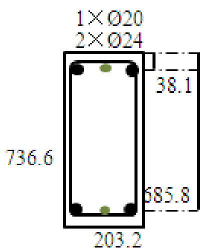
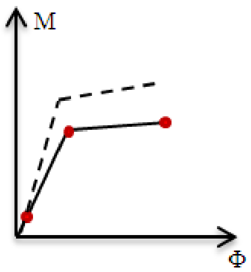
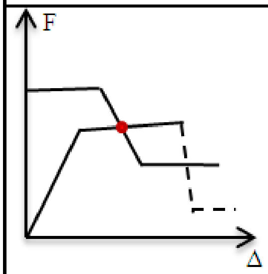
5.5.3. Developing Component Analysis Models and Global Structure Models

Similar to the component analysis models adopted in ASCE 41-13, it can be suggested that New Zealand Guidelines can be improved by adopting similar generalised force-deformation relationships for typical components. These generalised force-deformation relationships can be derived based on a great amount of analytical research (e.g. parametric study), numerical modelling and experiment work on the components, and the following should be taken account for: (more details shown in Table 5- 8)

- Interaction between flexure and shear
- Degradation of strength
- Residual strength
- Selection of critical parameters, and the influence of variation of these parameters to the force-deformation curves
- Specification of limit state criteria at component level, e.g. deformation or drift limits

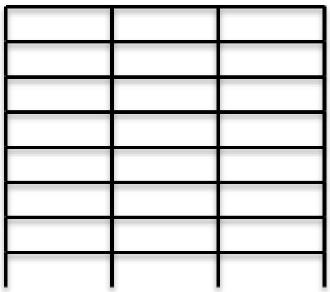
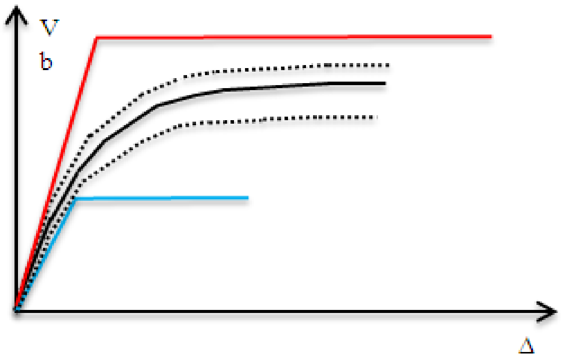
As illustrated in Figure 5- 9 in the beginning of Section 5.5, these component analysis models can be adopted directly in the nonlinear analysis programs, and then the numerical analysis can provide sophisticated predictions of structural responses in seismic events. Alternatively, these component analysis models can be applied as inputs to carry out simplified analytical analysis (i.e. the improved SLaMa) that does not requires numerical modelling. It is worth noting that analysis model for the secondary structural components and non-structural components should also be developed, since the behaviour of such components, and the interactions between such components and the primary structural components, may significantly influence the overall response of the structure in a seismic event.

Table 5- 8: Description of component model for beam, column, joint, wall and other components

Paramter Sets	Capacities	
	Momen-Curvature Model	
Set 1:  Selected representitave, section sketch, material properties, length, heigt, width, slenderness ratio, longitudinal reinforcing details and ratio, transverse reinforcing details and ratio, confinement, axial load ratio, etc.		Cracking moment and curvature, yielding moment and curvature, ultimate moment and curvaturr, bi-linear assumption or more complicated assumptions of the curve shape, influence of axial load, specification of limit states on component basis and defined as curvature, etc.
	Force-Deformation Model	
		Yielding force and deformation, ultimate force and deformation, shear envelope (i.e. interaction between flexure and shear), strength degradation, residual strength, assumptions of the curve shape, limit states on component basis and defined as deformation, etc.

It is suggested that not only the component analysis models, but also global structure models can be adopted in New Zealand Guidelines. Such global structure models (Table 5- 9) can be established based on a great amount of research work (including analytical, numerical and experimental) done for a large number of reinforced concrete building cases. The reinforced concrete building representatives to be studied should cover all the common reinforced concrete building types, as shown in Table 6- 9 in Section 6.3.2. For each of the building type, generalised pushover curves should be computed, and the changes of curves due to the variations of critical parameters are to be clarified (i.e. should be a set of parameters with properly defined variation ranges). With the adoption of such global structure models in seismic assessment, without involving complicated calculation and numerical modelling, the response of a reinforced concrete building can be directly predicted by referring to the generalised pushover curve of the building representatives that have the most alike properties or characteristics to the building case under assessment, even when only very limited information of the building to be assessed is available.

Table 5- 9: Description of global model

Parameter Sets	Capacities
<u>**Set 1:**</u> <u>**(one of the building classes)**</u>  Selected building representatives or established hypothetical structures, building year, number of floors, height, material properties, component properties and strengths (beams, columns, joints, walls, foundation, etc., may include secondary structural components and nonstructural components), subassembly strength of hirarchy, foundation-soil interaction, geotechnical issues, etc.	 Averaged pushover curve with varying ranges due to material strength uncertainties and component strength uncertainties, computation of upper and lower bounds depending on mechanisms The computation of the curves from: - Study of building representatives (i.e. building case studies from database) - Hypothetical structures With critical paramters varying in certain ranges (i.e. parametric study) - Application of by-hand analytical method - Application of numerical modelling

In addition, for each set of parameter ranges of one building type, lower and upper bounds of the generalised capacity curve can also be computed following the procedure in Section 5.5.1.

More details regarding the idea of adopting the component analysis model and global structure model are discussed in the following chapters. Future researches and investigations are required to compute these models, and also the applicability of the models should be further investigated.

5.6. Discussion

Table 5- 10 provides a summary of comparison among all the simplified analytical methods discussed in the previous sections. These procedures include:

- The current SLaMa in NZSEE 2006 (Section 4.6)
- The improve SLaMa with Evaluation of Strength Hierarchy (Section 5.4) and Determination of Lower and Upper Bounds of Lateral Load Capacity (Section 5.5.1)
- The improved SLaMa with Evaluation of Strength Hierarchy and Portal Frame Method (Section 5.5.2)
- The adoption of Component Analysis Models and Global Structure Models (Section 5.5.3)

It should be noticed that the discussion excludes the procedure involving numerical modelling with application of component models.

Table 5- 10: Comparison among the discussed simplified analytical methods

Approach	Current SLaMa from NZSEE 2006	Improved SLaMa with Evaluation of Strength Hierarchy and Determination of Lower and Upper Bounds of Lateral Load Capacity	Improved SLaMa with Evaluation of Strength Hierarchy and Portal Frame Method	Adoption of Component Analysis Models and Global Structure Models
Inputs	Component properties and strength capacities (Section 5.1-5.3)	Component properties and strength capacities (Section 6.1-6.4)	Component properties and strength capacities (Section 6.1-6.4)	Component analysis models and global structure models
Procedures or Tools	Analytical	Analytical	Analytical	Analytical
Outputs	Bilinear approximation of pushover curve	Possible ranges of pushover curve	Pushover curve with sequence of mechanisms shown	Pushover curve with varying ranges
Advantages	Simple	Simplest among the four procedures and provide a quick preliminary estimation	Most sophisticated among the four procedures	Can be the most convenient and efficient procedure, and can provide good results with acceptable varying ranges
Deficiencies	Highly potential to lead an overestimation of capacity, i.e. the correct mechanism tends to be missed, which makes the method infeasible in practice	Provide least accurate result, i.e. only the lower and the upper bounds	Most complicated among the four procedures because of the large amount of hand calculation, which is not favourable in practice	The establishment of the models requires great amount of research effort

The determination of the upper and lower bounds of lateral load capacity only provides a preliminary prediction of the structure response in a seismic event. Therefore, this approach should be applied together with other approaches, offering a way to check the assessment results.

By including evaluation of strength hierarchy and applying Portal Frame Method, a pushover curve with sequence of mechanisms can be computed. Though this approach can provide the most sophisticated assessment results compared to the other analytical approaches, the comparatively large amount of hand calculation work makes this method inapplicable in practice. Besides, there are some issues that should be addressed, which have significant influences on the accuracy of the outcomes:

- As discussed in Section 5.5.2, an inverted triangular lateral force profile is assumed, which is appropriate as the real force profile should be proportional to the displacement profile of the structure.
- As discussed in Section 5.5.2, the influence of some structural components and all nonstructural components cannot be accounted for in determining lateral load capacity and local displacement capacity.
- As discussed in Section 5.5.2, the deformation due to shear type of mechanisms is not properly calculated in the approach.
- Criteria for limit states are not clearly defined. Clarification is required regarding the criteria defined at material level (e.g. material strain limit), at component level (e.g. component deformation/drift limit), or at global level (e.g. interstorey drift limit).

The approach, in which the component analysis models and the global structure models are adopted, as discussed in Section 5.6, requires great research effort to establish such component and global structure models.

Considering the comparison among different approaches and the constraints of each approach, future researches and investigations are definitely required. It is worth noting that the validations of the improvements suggested should be achieved by: (1) correlation between results from the improved approaches and the observed damages; (2) comparison between results from the improved approaches and from numerical modelling, which are shown in Chapter 7 and 8.

5.7. Wall

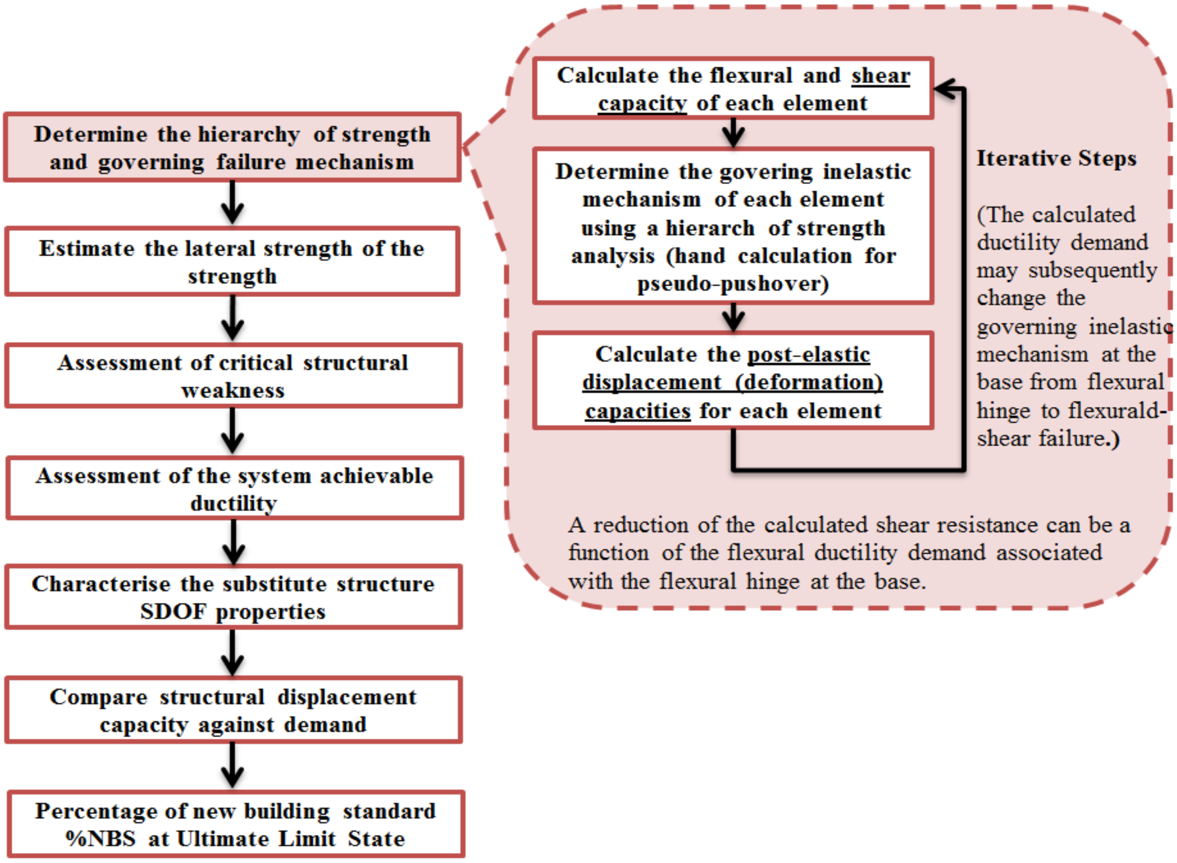


Figure 5- 22: Procedure of simplified displacement-based seismic assessment of a reinforced concrete shear wall building (summarised from Displacement-based Seismic Assessment: Practical Considerations, Kam, W.Y., Akguzel, U., Jury, R., Pampanin, S., 2013)

An overall view of SLaMa procedure applied to shear wall structures is given in Figure 5- 22, and an example (i.e. the hypothetical wall system shown in Figure 5- 23) was given in the paper *Displacement-based Seismic Assessment: Practical Considerations* to explain how the approach works.

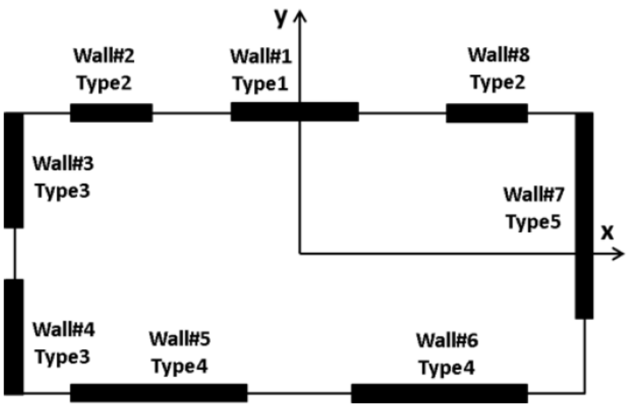


Figure 5- 23: Hypothetical wall structural plan for DBA example (internal gravity frames not shown)

As suggested in the Paper, after obtaining general understandings of the critical structural wall sections from structural drawings and preliminary estimation of the potential plastic zones, axial-flexural and shear capacities of these wall sections can be calculated. As for the determination of flexural capacities and curvatures of structural walls (i.e. probable flexural strengths), the conventional moment-curvature analyses can be applied, shown in Figure 5- 24. As for the determination of shear capacities of structural walls, (i.e. probable shear strengths) can be calculated using conventional shear assessment equations from NZS3101: 2006.

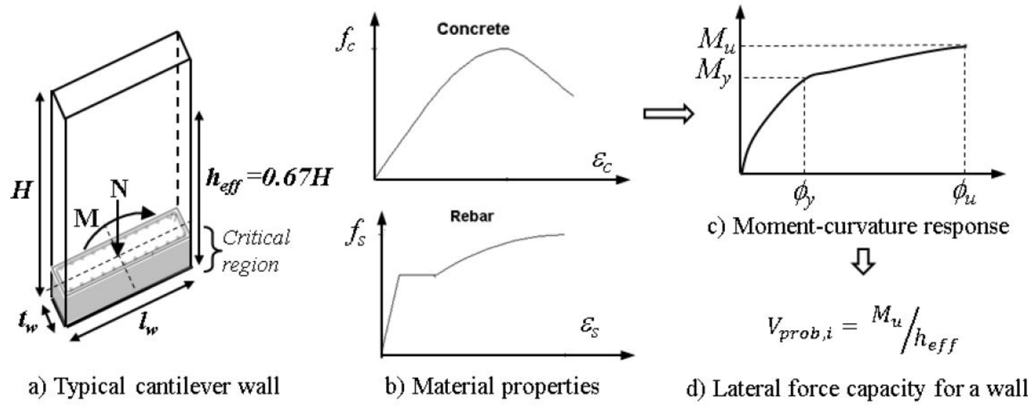


Figure 5- 24: Section analysis of a wall element

By comparing the determined probable base flexural capacities to the multiples of shear capacities and effective wall heights, the governing inelastic mechanisms can then be determined. Thereby, the achievable base shear should be the minimum of$\varphi M_u/H_{eff}$ and φV_u. It is worth noting that it has been recommended by Priestley *et al. (2007)* that the degradation of shear resistance as a function of the flexural ductility demand associated with the flexural hinge at the base should be accounted for in assessment. Therefore, as shown in Figure 5- 22, the previous steps may be iterative as the calculated curvature ductility demand may subsequently change the governing inelastic mechanism. Corresponding to the determined mechanism, yield displacement, plastic hinge length, plastic curvature, plastic displacement and ultimate displacement capacity at the effective height of the building can then be calculated. With the base shear and displacement capacities determined, bilinear push-over curves for the structural walls can be plotted. The lateral strength of the building in each principal direction then can be computed by super-positioning of all walls' base strength contributions at the critical displacement capacity, and specific checks of critical structural weaknesses (e.g. critical load path, horizontal diaphragm-to-wall, wall foundation, inelastic torsion stability from plan irregularity or the amplification of torsion effect due to the accidental eccentricity for elements at the edges, etc.) should be performed in order to ensure that the determined mechanisms can sustain.

At last, %NBS can be calculated by comparing the achievable ductility of the system (i.e. μ_{system}= ultimate displacement capacity/ yielding displacement) to the ductility demand that is represented by the structural ductility limit for the available detailing. For instance, if the reinforced concrete wall has non-ductile detailing such as plain bar lap splices, unconfined boundary ends, etc., a relatively conservative ductility limit of 2.0 should be applied.

6.1. Building Database

CHCH CBD Building Database (Kam. *Et.al.*), shown in Figure 6- 1, records basic building and damage information for 3620 buildings in Christchurch CBD, among which 831 are reinforced concrete buildings. The basic building information includes structural type, year built, location, building name, etc., and the damage information includes tagging, percentage of damages, severity of damages to structural and non-structural components, site hazards, etc.

	C	D	E	F	G	H	I	J	K	N	O	P	Q	R	
1	UoC Study	RFSNum	Nun	primary building cons	years bui	Buildin	usabili	Street Name	Situation Address	bldg d	business_name	demo_	demo_	sitehz_	s
2		75018616	1	7- Tilt-up Concrete	1970-1979	Green	G: Green I	Aberdeen St	12 ABERDEEN ST	None	NULL	NULL	NULL	1- Minor/N	1
3		75018764	1	8- Timber Frame	1900-1909	Green	G: Green I	Aberdeen St	18 ABERDEEN ST	None	NULL	NULL	NULL	1- Minor/N	1
4		75084353	1	6- Steel Frame	1980-1989	Green	G1: Occup	Aberdeen St	22 ABERDEEN ST	2-10%	NULL	NULL	NULL	4- Unknow	2
5		75009882	1	7- Tilt-up Concrete	1980-1989	Red	R: Level 1	Aberdeen St	24 ABERDEEN ST	2-10%	Rubber Plus	NULL	NULL	2- Moderat	3
6		75014890	1	6- Steel Frame	1970-1979	Green	G: Green I	Acton St	10 ACTON ST	None	CHCH Auto AIR	NULL	NULL	1- Minor/N	1
7		75014872	1	6- Steel Frame	1970-1979	Yellow	Y: Yellow	Acton St	12 ACTON ST	None	Peter Geary Motors	NULL	NULL	1- Minor/N	1
8		75014872	1	6- Steel Frame	1970-1979	Yellow	Y1: Short	Acton St	12 ACTON ST	2-10%	Peter Geary Motors	NULL	NULL	2- Moderat	1
9		75014930	2	1- Concrete Frame	2000-2009	Yellow	Y: Yellow	Acton St	4 ACTON ST	2-10%	A new standard For living	NULL	NULL	1- Minor/N	1
10		75014908	1	6- Steel Frame	1970-1979	Green	G: Green I	Acton St	6 ACTON ST	0-1%	Cartune	NULL	NULL	1- Minor/N	1
11		75014899	1	2- Concrete Shear Wall	1970-1979	Green	G: Green I	Acton St	8 ACTON ST	0-1%	Collosion Repair	NULL	NULL	1- Minor/N	1
12		75006870	4	4- RC Frame with Masonry	NULL	Green	G: Green I	Airedale Pl	16 AIREDALE PLCE	31-60%	NULL	NULL	NULL	Minor/Non	M
13		75022993	2	2- Concrete Shear Wall	NULL	Green	G: Green I	Allen St	34 ALLEN ST	0-1%	CPIT	NULL	NULL	1- Minor/N	1
14		75009782	1	2- Concrete Shear Wall	1980-1989	Green	G: Green I	Allen St	10 ALLEN ST	2-10%	NULL	NULL	NULL	1- Minor/N	1
15		75080044	2	6- Steel Frame	1950-1959	Green	G2: Occup	Allen St	11 ALLEN ST	None	Suzuki Building (workshop)	NULL	NULL	1- Minor/N	1
16		75011506	1	5- Reinforced Masonry	Various Ages	Green	G: Green I	Cashel St	265 CASHEL ST	None	Subway	NULL	NULL	NULL	N
17		75013423	2	7- Tilt-up Concrete	Various Ages	Green	G: Green I	Antigua St	202 ANTIGUA ST	0-1%	NULL	NULL	NULL	1- Minor/N	1
18		75080021	1	6- Steel Frame	1950-1959	Green	G2: Occup	Allen St	35 ALLEN ST	0-1%	NULL	NULL	NULL	1- Minor/N	1
19		75013500	1	8- Timber Frame	Various Ages	Green	G: Green I	Antigua St	272 ANTIGUA ST	None	NULL	NULL	NULL	1- Minor/N	1
20		75017217	1	1- Concrete Frame	1960-1969	Green	G: Green I	Antigua St	163 ANTIGUA ST	NULL	INTACTUNE	NULL	NULL	1- Minor/N	1
21		75017185	1	1- Concrete Frame	1970-1979	Green	G: Green I	Antigua St	165 ANTIGUA ST	NULL	NULL	NULL	NULL	1- Minor/N	1
22		75017164	1	4- RC Frame with Masonry	1960-1969	Yellow	Y: Yellow	Antigua St	169 ANTIGUA ST	NULL	Montreux Furniture	NULL	NULL	1- Minor/N	1
23		75017338	1	4- RC Frame with Masonry	2000-2009	Green	G: Green I	Antigua St	181 ANTIGUA ST	NULL	NULL	NULL	NULL	1- Minor/N	1
24		75018001	2	7- Tilt-up Concrete	1990-1999	Green	G: Green I	Antigua St	185 ANTIGUA ST	0-1%	NULL	NULL	NULL	1- Minor/N	1
25		75018091	2	6- Steel Frame	1970-1979	Green	G: Green I	Antigua St	187 ANTIGUA ST	0-1%	Proto Electronics	NULL	NULL	1- Minor/N	1
26		75018107	1	6- Steel Frame	1970-1979	Green	G: Green I	Antigua St	189 ANTIGUA ST	2-10%	NULL	NULL	NULL	1- Minor/N	1
27		75026055	10	1- Concrete Frame	NULL	Green	G: Green I	Armagh St	133 ARMAGH ST	0-1%	NULL	NULL	NULL	1- Minor/N	1
28		75068394	1	7- Tilt-up Concrete	1980-1989	Yellow	Y1: Short	Antigua St	198 ANTIGUA ST	2-10%	NULL	NULL	NULL	NULL	N
29		75000262	1	1- Concrete Frame	Various Ages	Green	G: Green I	Armagh St	147 ARMAGH ST	0-1%	NULL	NULL	NULL	Minor/Non	M
30		75009760	2	5- Reinforced Masonry	Various Ages	Green	G: Green I	Cashel St	309 CASHEL ST	2-10%	NULL	NULL	NULL	1- Minor/N	1
31		75015015	0	10- Others (Carpark)	2000-2009	Green	G: Green I	Antigua St	212 ANTIGUA ST	NULL	NULL	NULL	NULL	1- Minor/N	1
32		75012783	2	7- Tilt-up Concrete	1980-1989	Yellow	Y: Yellow	Antigua St	220 ANTIGUA ST	11-30%	NULL	NULL	NULL	2- Moderat	2
33		75014856	2	1- Concrete Frame	2000-2009	Green	G: Green I	Antigua St	226/228 ANTIGUA ST	0-1%	City Care	NULL	NULL	1- Minor/N	1
34		75000490	2	7- Tilt-up Concrete	1990-1999	Yellow	Y: Yellow	Antigua St	230A ANTIGUA ST	2-10%	NULL	NULL	NULL	4- Unknow	4
35		75068244	1	9- Un-reinforced Masonry	1980-1989	Red	R: Level 1	Antigua St	230B ANTIGUA ST	61-99%	NULL	NULL	NULL	1- Minor/N	1
36		75027820	5	1- Concrete Frame	1990-1999	Green	G2: Occup	Antigua St	235 ANTIGUA ST	2-10%	Christchurch Hospital Car Pa	NULL	NULL	1- Minor/N	1
37		75014156	1	8- Timber Frame	Various Ages	Green	G: Green I	Armagh St	257 ARMAGH ST	2-10%	NULL	NULL	NULL	1- Minor/N	1
38	2	75012865	2	6- Steel Frame	2000-2009	Yellow	Y1: Short	Antigua St	258 ANTIGUA ST	11-30%	CDHB CAP PARK	NULL	NULL	2- Moderat	1
39		75018537	2	8- Timber Frame	Various Ages	Green	G: Green I	Armagh St	289 ARMAGH ST	2-10%	NULL	NULL	NULL	1- Minor/N	1
40		75018604	1	8- Timber Frame	Prior to 1880	Green	G: Green I	Armagh St	313 ARMAGH ST	0-1%	NULL	NULL	NULL	1- Minor/N	1
41		75013465	2	8- Timber Frame	1950-1959	Green	G: Green I	Antigua St	278 ANTIGUA ST	2-10%	Cardio/Respitory Outreach	NULL	NULL	1- Minor/N	1
42		75013689	2	4- RC Frame with Masonry	1960-1969	Green	G: Green I	Armagh Crt	1 ARMAGH CRT	0-1%	NULL	NULL	NULL	1- Minor/N	1
43		75081192	2	4- RC Frame with Masonry	1960-1969	Yellow	Y1: Short	Armagh Crt	10 ARMAGH CRT	11-30%	Armagh Court	NULL	NULL	1- Minor/N	1
44		75010913	2	5- Reinforced Masonry	Various Ages	Green	G: Green I	Cashel St	320 CASHEL ST	None	Indigo and nova, EBOS	NULL	NULL	1- Minor/N	1
45		75081136	2	4- RC Frame with Masonry	1960-1969	Yellow	Y1: Short	Armagh Crt	3 ARMAGH CRT	11-30%	Armagh Court	NULL	NULL	1- Minor/N	1

Notes | PivotTable | MAINData (Refined) | RCStatisticsJune2011(SHARON) | Foglio2 | URM | Damage_RC_Fran

Ready 100%

Figure 6- 1: EXCEL spreadsheet of CHCH CBD Building Database (Page 1)

The buildings from the Database can be categorised into four types of reinforced concrete structures – frames, walls, frames with masonry infill and tilt-up structures. For each of the four reinforced concrete building types, Table 6- 1 shows the number of buildings that were tagged with green, yellow and red after the earthquake sequence, and the corresponding proportions. The number of buildings under each building age category (i.e. pre 1930s, 1940s, 1950s, 1960s, 1970s, 1980s, 1990s-2010s, unknown) and building height category (i.e. in terms of number of stories, 1 storey, 2 storey, 3-4 storey, 5-8 storey, 9+ storey) for the four structural types is shown in Figure 6- 1. The information conveyed by Table 6- 1 and Figure 6- 1 is useful for the determination of building typologies.

Table 6- 1: Statistics of reinforced concrete buildings in CHCH CBD Building Database

Type of Construction	NZSEE Building Safety Evaluation Tagging		
	Green	Yellow	Red
Reinforced Concrete Frames	179 (50.3%)	101 (28.4%)	76 (21.3%)
Reinforced Concrete Shear Wall	44 (48.4%)	29 (31.9%)	18 (19.8%)
Reinforced Concrete Frames with Masonry Infill	98 (47.1%)	86 (41.3%)	24 (11.5%)
Tilt Up Concrete	120 (68.2%)	40 (22.7%)	16 (9.1%)

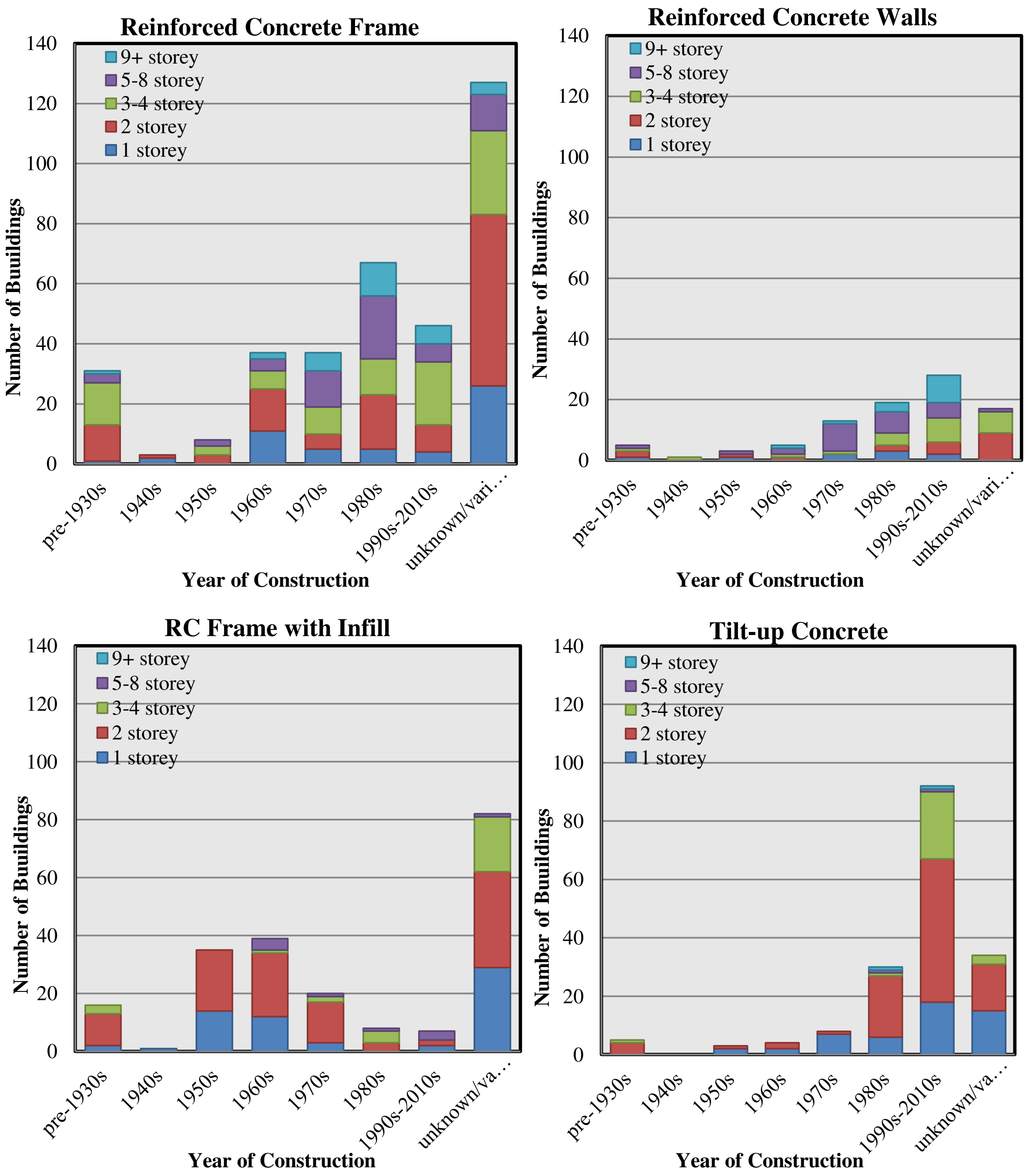


Figure 6- 2: Age and number of storey statistics for the four typical reinforced concrete building types

More detailed damage information included in the Database, together with the statistics, is shown in Section 6.2. To be concise, in the Database, damage ratio, tagging information, the observed damages to structural and non-structural components, and hazards due to surrounding buildings, soil condition, slop, etc. are recorded.

The Database has been refined for 154 reinforced concrete buildings on the basis of the information provided in University of Canterbury Research Report on Observed Earthquake Damage of Reinforced Concrete Buildings in the Christchurch CBD on the 22 February 2011 Earthquake (Pampanin*et.al.* 2011), as shown in Figure 6- 3 and Figure 6- 4.

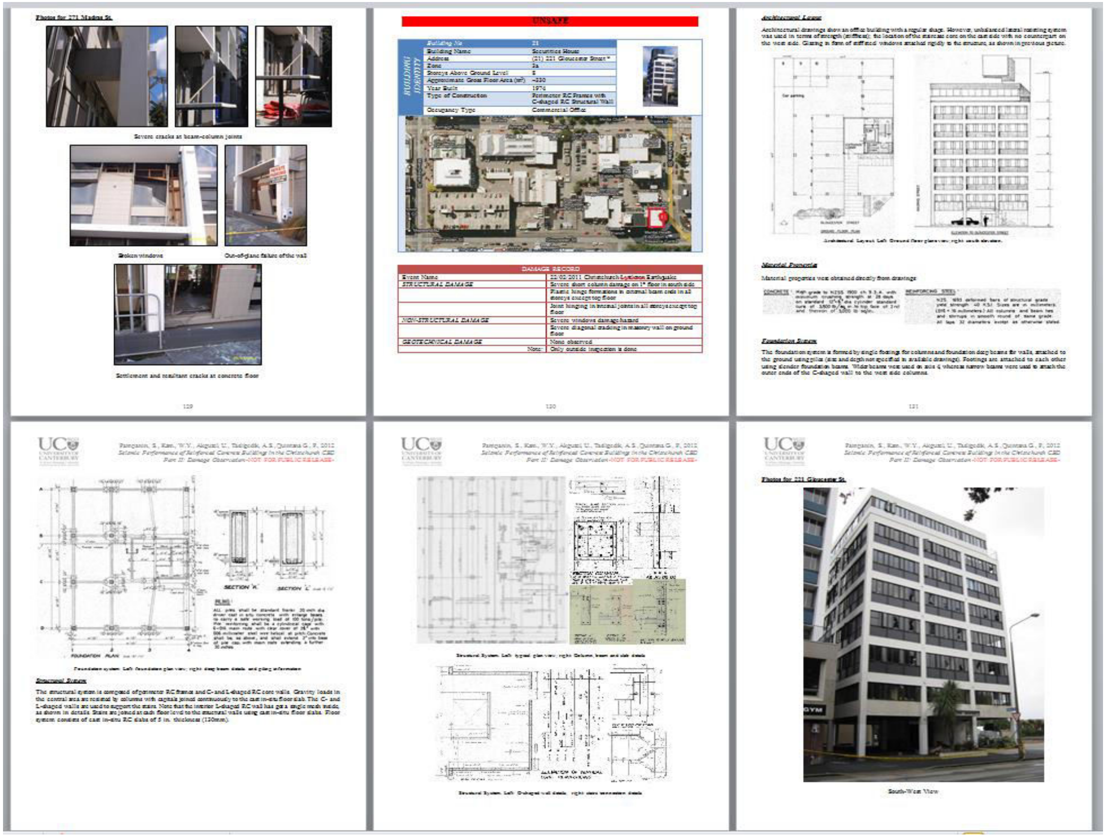


Figure 6- 3: Research Report on Observed Earthquake Damage of Reinforced Concrete Buildings in the Christchurch CBD on the 22 February 2011 Earthquake (Pampanin et al. 2011), the Example in the Figure is Securities House, with Building Data, Damage Data and Detailed Structural Drawings

1	Based on BSE (Building Safety Evaluation) Level 1 and Level 2 Evaluation											
2	F=frame	SW=shear	RMW=reinforced			COL=column						
3		CIW=concrete infilled wall, IW=infilled wall,	RF=reinforced floor			J=joints						
4		ST=stairs	M=masonary			GF=ground floor						
5		U=unreinforced	C=concrete			F=frame						
6	Building Damage Statistics In Christchurch											
7	No	Building Name	Building Address	Year Build	Storey	Type of Construction	Approx. Gross Floor Area	Failure/Damages	Structural Damage	Non-structural Damage	Geotechnical Damage	Note
8												
9												
10	1		35 Cambridge Tce *	1960-1969	6	RM	300-400	RMW	Hollow M failure; No grout;	Windows broken	None; liquefaction;	No futher inside inspect
11	2	Warren and Mahoney	131 Victoria Street	1960-1969	2	RCF with SW	150	None	None; treat from failure or colla	None	None	
12	3	Fidelity House	167 Victoria Street	1930-1970	4	RM	400	RMW/SL/COL	RM(block) shear cracks; RMW	Ceiling; partition wall; wind	None	
13	4	Vaughan Antiques	54 Salisbury Street	1950-1959	2	RCF with M Infill	100	COL/BCJ/UMW	COL ends flexural cracking (at C	F infill step cracking; ceiling	20mm settlement at left side	
14	5	Amuri Courts	263 Durham Street *	1978-1986	5	Precast RCF, RC SW, R	7000	F/SW/B/COL	F flexural cracks; SW and deep t	IW diagonal cracking and sli	Liquefaction; transverse mo	Discontinuous SW and c
15	6	ROMA Caffe	176 Oxford Terrace	1930-1939	3	UM	150	UM (brick)	UM 5mm vertical crack; UM di	Side wall seperated from ma	None	Overhead parapet dama
16	7	Brannigans	86 Gloucester Street	1980-1989	10	CF	800	B	B (at 2nd FL) exposed bars at m	Windows; adjacent buidling	None	B ends was curred/shap
17	8	TV NZ	198 Gloucester Stree	Pre1980s	4	CF	250	COL/J/SW/ST	COL and J shear failure (north f	Partition walls (gypsum lin	None	COLs with buckling lon
18	9	Pacific Tower (HCG O	123Victoria Street *	1980-1989	7	RCF with SW	225	SW/ST	SW severe damages; ST connect	Ceiling (place connected wi	None	
19	10	Crowne Plaza	70 Kilmore Street *	1980-1989	10	RCF with SW	1000	SW/BCJ/ST	SW severe damages at ST and se	ST support minor spalling	Liquefaction (moderate)	
20	11		822 Colombo Street	1960-1969	4	RCF with M Infill (bloc	400	COL	COL severe crushing and spallin	Windows (severe at GF); w	None	
21	12	St. James Court	77 Gloucester Street	1990-1999	3	RCF with SW	400	SW	SW minor horizontal and diagon	None	None	
22	13	Clarendon Tower	78 Worcester Street *		17+1B	RCF with SW	900	ST/SL	ST collapse (between Level 7 an	Windows and glass (GF); el	Ground slab separation	Structural damage detail
23	14	Public Trust Office	152 Oxford Terrace	1920-1929	5	RCF with SW; URM in	600	SW/URM	SW (with windows) minor shea	Infills (at ST) moderate slid	None	
24	15	Bank ANZ Cathedral S	121 Hereford Street	Pre70s	4	RCF	1000	COL	COL (GF) severe crushing and t	Windows severely broken;	None	Only GF has been inspe
25	16	Holy Grail	88 Worcester Street	1930-1939	4	RCF with M Infill	600	None	None	Partition walls minor separa	None	
26	17	Gough House	90 Hereford Street	1930-1939	4	RCF with M Infill	600	COL/B	COL (front) severe shear failure	MIW corner crushing, diago	None	Only outside inspection
27	18	Wine Room	118 Hereford Street	1920-1929	4	CF	100	COL/F	COL (2nd FL) severe plastic hin	Parapets (top of building) p	None	Only outside inspection
28	19	Wendy's	119 Hereford Street		4	RM (blocks)	120	RM	RMW (GF) failed with bars rev	Not inspected from inside.	None	Reinforcement of the rei
29	20	Grenadier House	271 Madras Street *	1960-1969	5	RCF with M Infill	400	BCJ	Leaning severely to south (wall	Windows (south, severe)	Settlement severely on sout	Only outside inspection
30	21	Securities House	221 Gloucester Stree	1974	8	RCF(perimeter)&C-shap	330	COL/B/J	Short COL damage (1st FL, sou	Windows severe damages; N	None	Only outside inspection
31	22	Camelot Hotel	66 Cathedral Square	1970-1979	7	CSW	900	None	None observed	Partition walls minor separa	None	
32	23	Poplars Apartments	82 Chester Street Eas	1950-1959	6	CF with CSW	500	COL/SW	Leaning severely to Madras St;	Windows (moderate)	Liquefaction (moderate, west)	
33	24	Westpack Building	107 Armagh Street	1920-1929	2	RCF with M Infill	Commercial/Off	SL	Old and new extension severely	Parapet (moderate) damage;	Settlement/uplift (moderate) between the old and the	
34	25	Souvenirs	730 Colombo Street	1930-1939	3	RCF with M Infill	Commercial/Off	None	None observed	Ceiling (moderate); M brick	None	
35	26	SBS Tower	180 Manchester Stre	1960-1969	9	CSW, SF encased in C	900	B/Couple-B/SL/COl	Coupling B (E, W, outside all Fl	Veranda (GF) severe overhe	None	The coupling B don't ha
36	27	New Zealand College of	149-151 Hereford St	1930-1939	3	URM, CSW (added later	150	Brickwall/URM/ST	Brick wall (severe, west) collaps	Boards (gypsum, minor) se	None	

Figure 6- 4: EXCEL Spreadsheet of the Refined Database for 154 RC Buildings (Page 1)

As shown in Figure 6- 5, the frame-type structures take the largest proportion in the Refined Reinforced Concrete Database – up to about 86%, and the frame-type structures can be categorised into 6 sub-types: bare frame, frame with shear wall, frame with flat slab, frame with infills, frame with tilt-up concrete, and precast frame. It is illustrated in Figure 6- 5 that the frame with infills sub-type takes the largest proportion (73%) of pre-70s frame buildings, while the frame with shear wall sub-type takes the largest proportion (44%) of the post-70s frame buildings. There are significant increases in the numbers of precast frame buildings and frame with shear wall buildings after 1970s, and there is large reduction in the number of frame with infills buildings.

The damage statistics drawn from the Refined Database are much less reliable compared to those drawn from the Database, due the very small number of buildings included in the refined database. However, in the Refined Database, much more detailed information regarding observed damages to the components is included, and is discussed in Chapter 7 (building case studies).

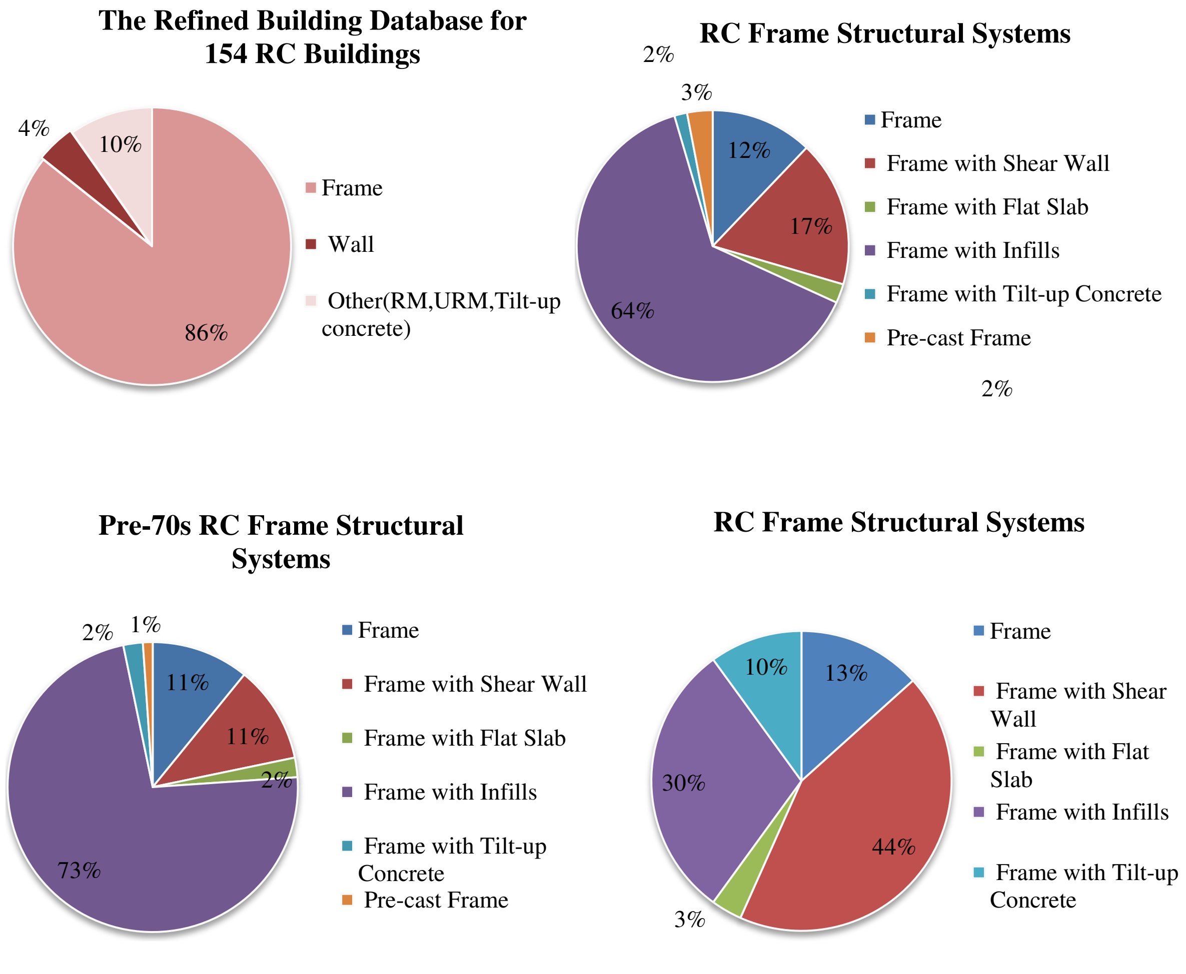


Figure 6- 5: (1) Common types of reinforced concrete buildings in the Refined Database; (2) Common RC frames types found in the Refined Database; (3) Pre-70s RC Frames; (4) Post-70s RC Frames

6.2. Observed Damages

Figure 6- 6, Figure 6- 7 and Figure 6- 8 illustrate the statistics that are drawn from CHCH CBD Building Database, and Figure 6- 9 shows the statistics drawn from the Refined Database. It is worth noting this section only provides information regarding damages, without comparison with the predictive results from seismic assessment.

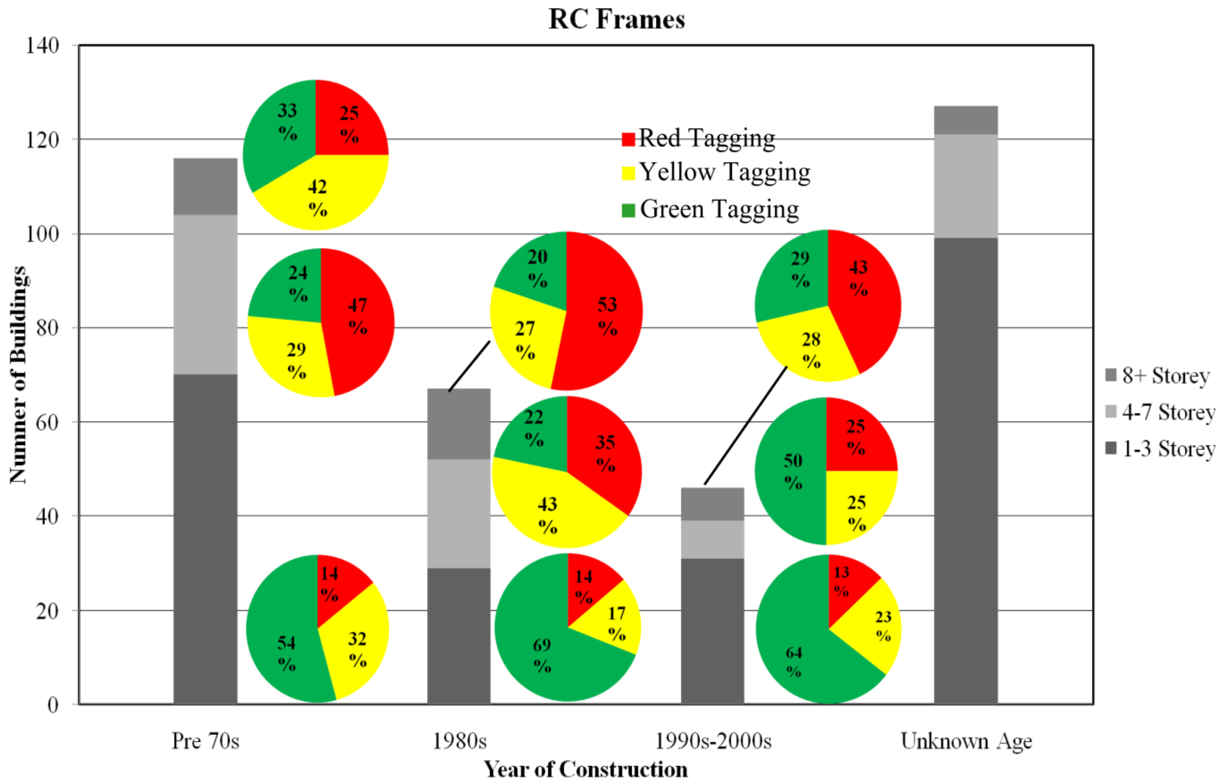


Figure 6- 6: Damage statistics summary: reinforced concrete frame buildings (low-rise, mid-rise, high-rise with red, yellow or green tagging)

As shown in Figure 6- 6, low-rise reinforced concrete frames appear to be safer compared to mid- or high-rise frames, with 54% of pre 70s (including 70s) frames found as green tagged, 69% of 1980s frames and 64% of 1990s-2000s frames. For pre70s frames, the mid-rise frames seem to be more vulnerable than the high-rise frames, as 47% of the mid-rise frames were red-tagged while only 25% of the high-rise frames were red tagged. For the post 70s frames, however, higher proportions (i.e. 53% for 1980s frames and 43% for 1990s-2000s frames) of the high-rise frames were red tagged compared to mid-rise frames (i.e. 35% for 1980s frames and 25% for 1990s-2000s frames).

For the reinforced concrete shear wall buildings, as shown in Figure 6- 7, there is a significant increase in the number of high-rise wall buildings since 1990s. Among these comparatively new high-rise wall buildings, 60% of them were red tagged after the earthquakes, indicating that severe structural deficiencies may exist in this type of building. For the reinforced concrete frames with masonry, the records show that no high-rise masonry infilled frames were included in the CHCH

CBD Building Database, and majority of the recorded infilled frames are low-rise, and significant damages were recorded to these low-rise infilled frames that were designed and constructed during 1980s.

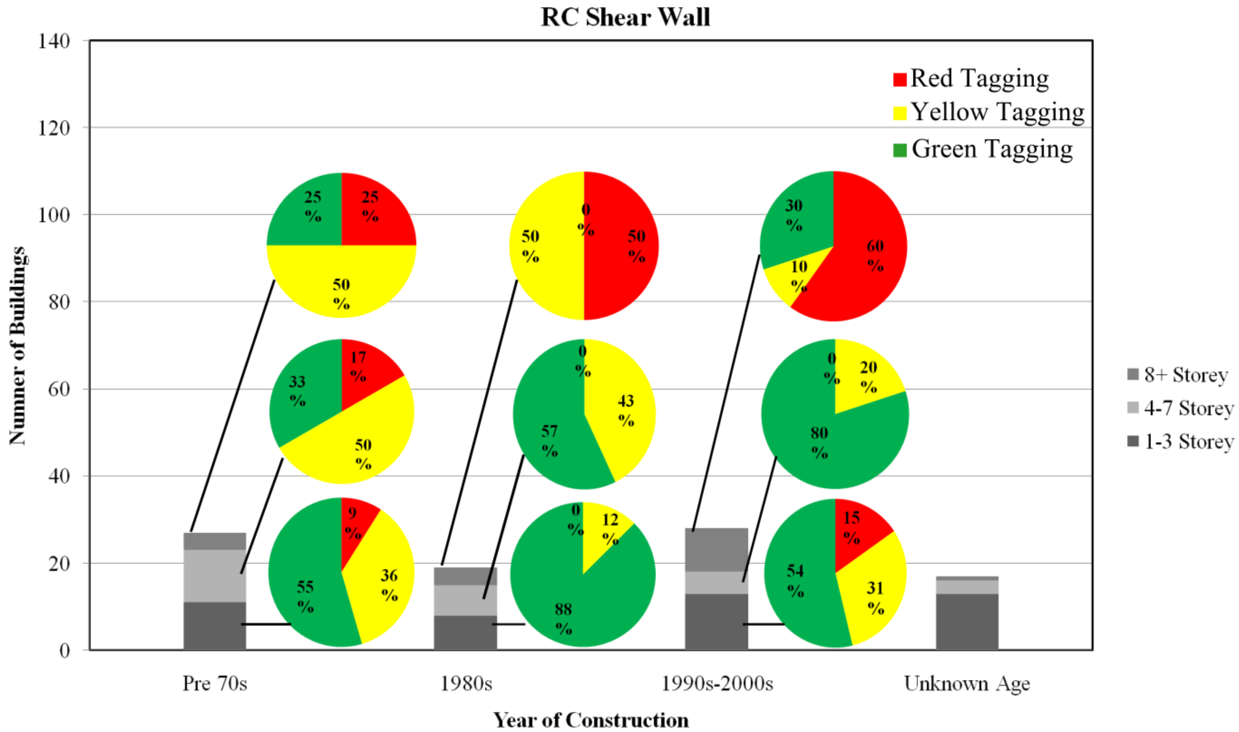


Figure 6- 7: Damage statistics summary: reinforced concrete shear wall buildings (low-rise, mid-rise, high-rise with red, yellow or green tagging)

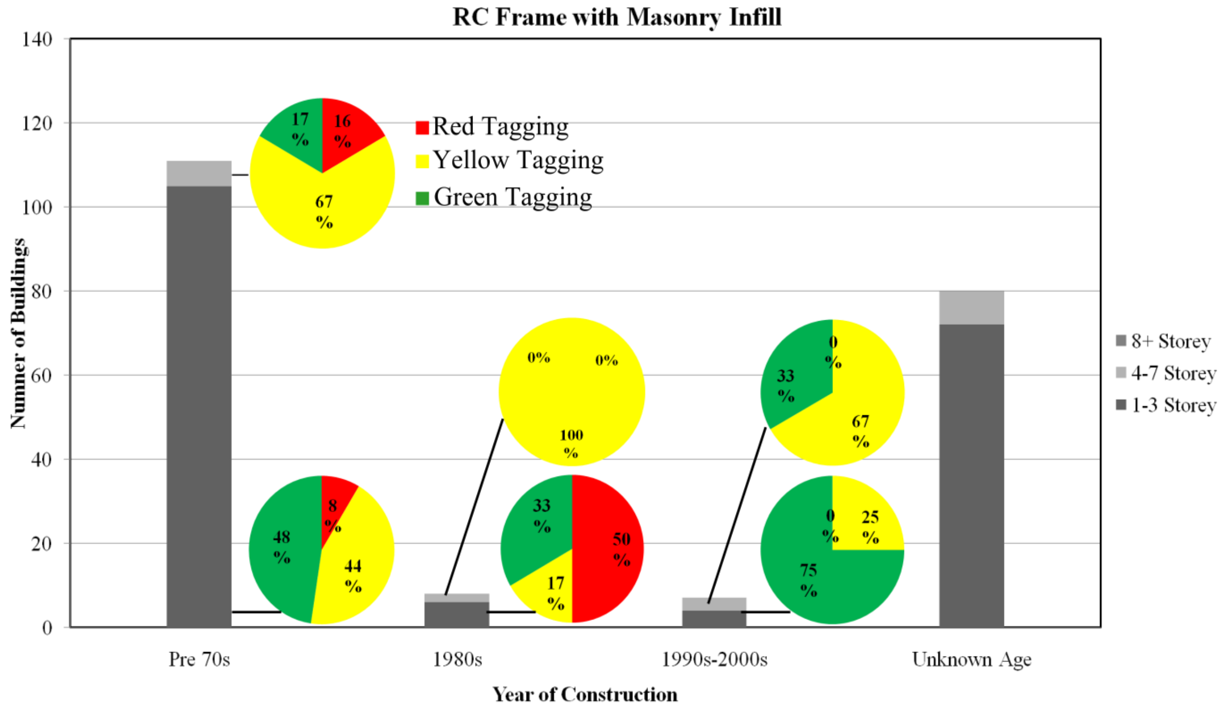


Figure 6- 8: Damage statistic summary: reinforced concrete frames with masonry (low-rise, mid-rise, with red, yellow or green tagging)

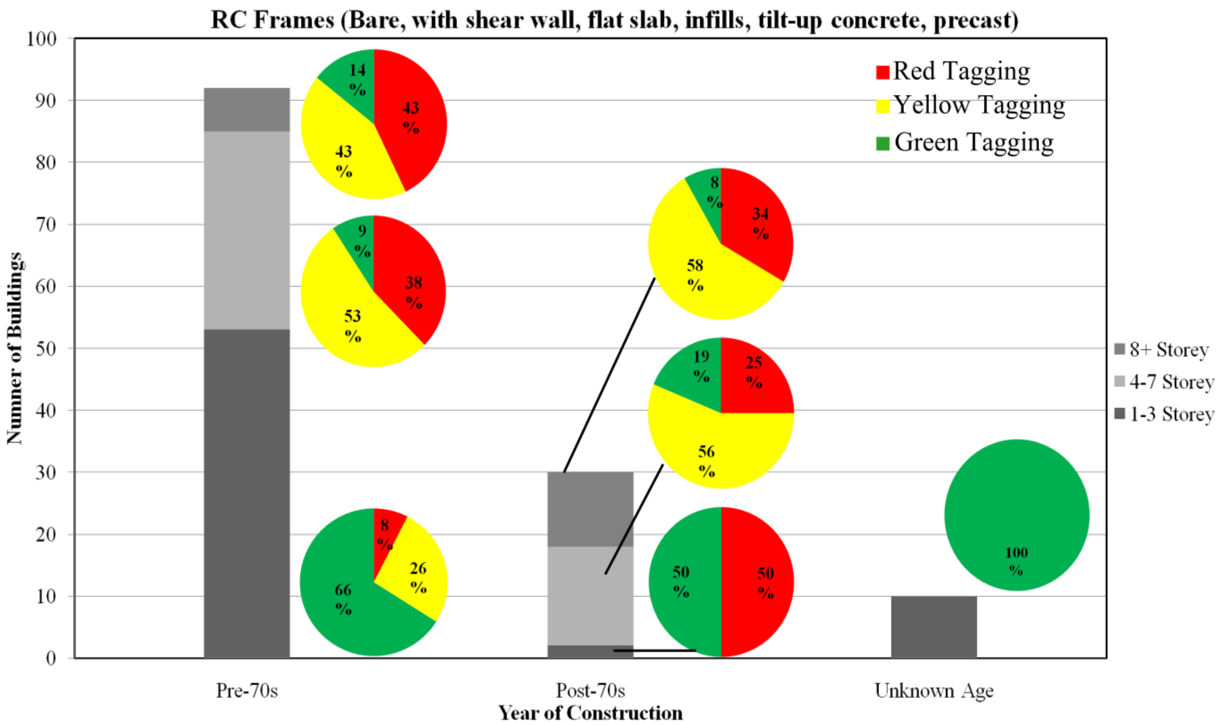


Figure 6- 9: Damage statistic summary for the refined building database: reinforced concrete frames (including bare frames, frames with shear walls, flat slabs, infills or tilt-up concrete and precast frames)(low-rise, mid-rise, with red, yellow or green tagging)

Figure 6- 9 illustrates the statistic drawn from the 154 buildings in the Refined Building Database, but it should be reaffirmed that the statistical results are not reliable due to the limited number of buildings in the Refined Database. Least damages were observed for the low-rise buildings that were constructed pre 70s.

6.2.1. Tagging

According to Building Safety Evaluation during a State of Emergency – Guidelines for Territorial Authorities (NZSEE, 2009), the post-disaster Building Safety Evaluation process endorsed by Department of Building and Housing involves three levels of assessment, shown in the following:

- Initial Assessment: a brief screening of the exterior of the building in order to identify any signs of imminent danger
- Rapid Assessment (Level 1 and Level 2): inspection through the building if access if permitted and to identify any signs of significant structural damage
- Detailed Engineering Evaluation (DEE): detailed evaluation of the building design, construction and potential response in seismic event, and determine strengthening or rehabilitation measures if required to meet selected performance level

In both CHCH Building Database and the Refined Building Database, tagging information is included for each inspected building from Initial Assessment or Rapid Assessment (Level 1 and Level 2). Table 6- 2 provides a summary of the definitions of the different colour tags under different assessment levels. As shown in Table, a Level 1 Rapid Assessment results in a building being tagged as Green, Yellow or Red, whereas a more detailed Level 2 Assessment includes further classifications into 6 grades (S. R. Uma, *et al.*).

Table 6- 2: Definition of different building colour tagging categories (S.R. Uma, etal.)

Level of Assessment	Colour Tagging	Definition
Level 1 Rapid Assessment	Red I	Unsafe, and do not enter. Further assessments or evaluation required before any use
	Yellow (Y)	Restricted use; Safety concerns; parts may be off limits; entry only for short periods of time for retrieving important goods
	Green (G)	Inspected, and apparently ok; but may need further inspection or repairs
Level 2 Rapid Assessment	Red 1 (R1)	Significant damage repairs strengthening possible
	Red 2 (R2)	Severe damage demolition likely
	Yellow 1 (Y1)	Short term entry only
	Yellow 2 (Y2)	No entry to parts until secured or demolished
	Green 1 (G1)	Occupiable and no immediate further investigation required
	Green 2 (G2)	Occupiable and repairs required

6.2.2. Percentage of damage

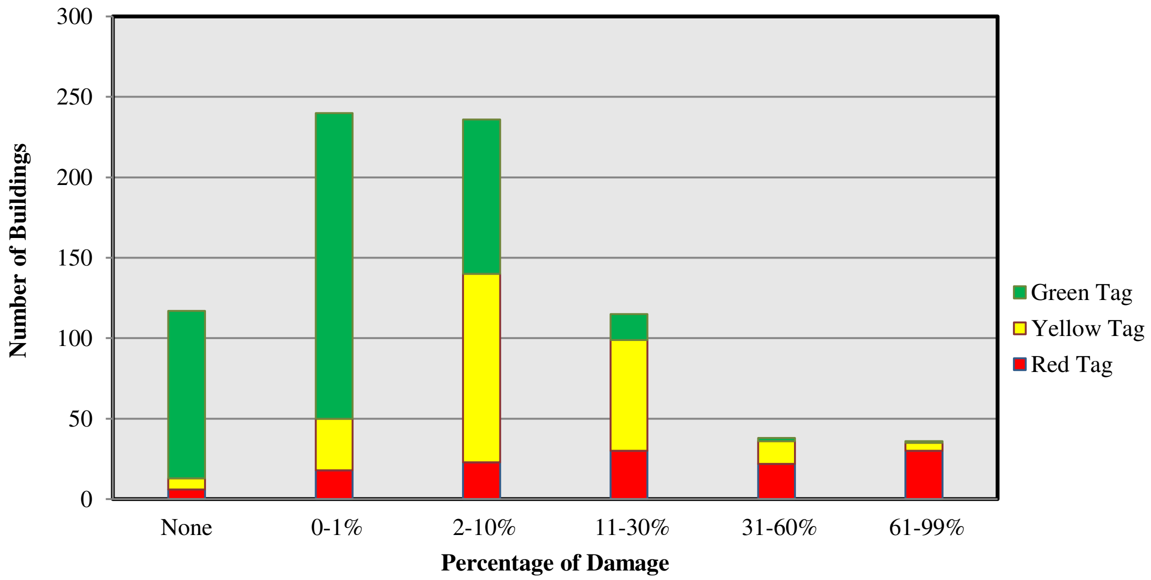


Figure 6- 10: Tagging of reinforced concrete buildings (in CHCH CBD Building Database) of the six percentage of damage categories (i.e. none damage, 0-1%, 2-10%, 11-30%, 31-60%, and 61-99%)

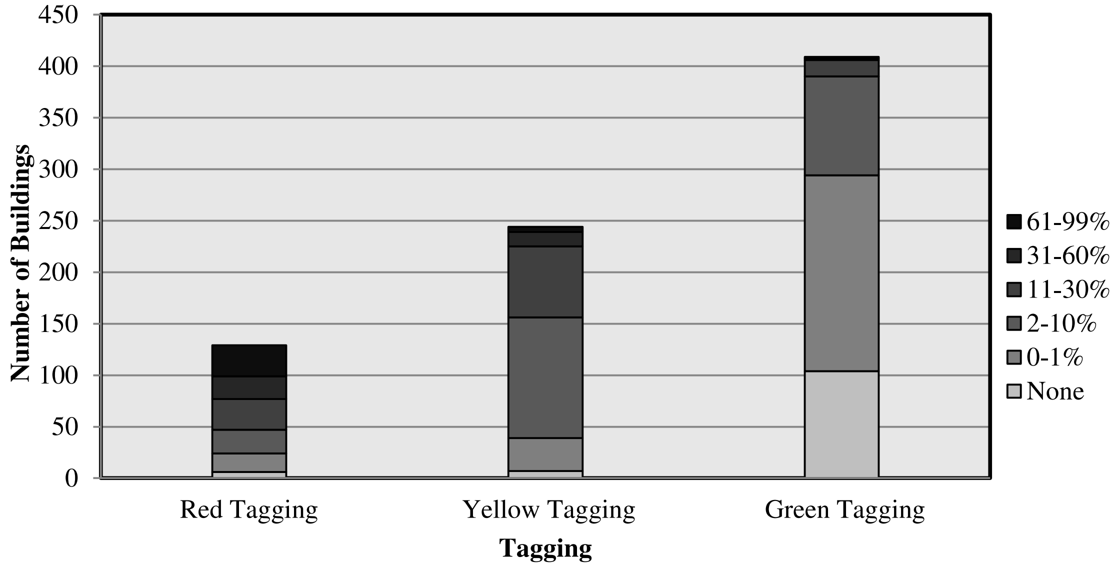


Figure 6- 11: Percentage of damage of reinforced concrete buildings (in CHCH CBD Building Database) of the three tagging categories (i.e. red tagging, yellow tagging, and green tagging)

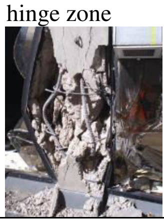
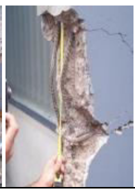
From Figure 6- 10 and Figure 6- 11, no definite correlation between tagging and percentage of damage can be drawn. In generally, the following trends are summarised:

- In the low percentage of damage ranges (e.g. 0~around 10%), buildings with green tagging take the largest proportion; in the mid percentage of damage ranges (e.g. around 10%~30%), buildings with yellow tagging take the largest proportion; in the high percentage of damage ranges (e.g. 30%~99%), buildings with red tagging take the largest proportion.
- However, few buildings with red tagging have been found in the low percentage of damage categories. Most of these buildings did not have severe structural or non-structural damages, but had hazard caused by the adjacent structures and geotechnical issues. It has been also found that very few buildings with green tagging are in the high percentage of damage categories. This is more likely due to the judgement being biased during evaluation of the buildings.
- In the red tagging building category, buildings with percentages of damage ranging from 0 to 99% are found. As mentioned in previous, the reason for a building is tagged as “red” does not necessarily indicate that the building was subject to devastating damages during earthquake. Other possible reasons can be hazard from adjacent structures (e.g. leaning of the nearby building, collapse of the nearby building, etc.) or geotechnical issues (e.g. sinking and settlement of the land, liquefaction of the land, etc.).
- In the yellow tagging building category, no buildings from the 61~99% percentage of damage range has been found. And in the green tagging building category, no buildings from the 31~99% percentage of damage range has been found.

6.2.3. Structural Damages

As shown in Table 6- 3, the critical observed structural damages are summarised. The 154 buildings in the Refined Database are classified in to two general categories: pre 70s (including 70s) buildings and post 80s buildings. For the buildings designed and built pre 1970s or during 1970s, due to the reason that the capacity design principles were not widely applied in practice, columns sidesway mechanisms (i.e. soft storey mechanisms, strong beam-weak column mechanisms) were observed. Therefore, the beams were found to generally subject to flexural types of damages while the columns and joints were generally subject to shear types of damages. The wall sections designed and built in this period, lacked appropriate reinforcing detailing along wall height, were observed to buckle out of plane and crush at boundary zones. In the period of post 1980s, together with the development of construction techniques and more knowledge of capacity design, materials, components, etc., a lot more buildings were designed with very complicated structures (e.g. hanging over components/parts, transferring components/parts, etc), resulting in significant irregularity vertically and horizontally. More damages due to torsional actions or torsional-flexural/shear mixed actions were observed on components. These damages have aroused high attention to design to resist torsion effects and assessment of response of components due to torsion actions. The guidelines should be subject to improvement and refinements with the outcomes of researches regarding this issue.

Table 6- 3: Structural damages observed and summarised from the refined building database

Component or Global Structure	Damages	
	Pre 70s (including 70s)	**Post 80s**
Global Structure	Soft-storey mechanism with severe damages found on columns and joints because of the lack of capacity design in old days	Damages due to plan and vertical irregularity
Beams	Flexural or shear types of damages found on beams due to poor confinement and shear reinforcement in beams, especially at end region and splice region.   Slip of reinforcing bars due to the use of plain round reinforcing bars in old days	Beam elongation triggers failure of diaphragm  
Columns	Buckling or shear failure of column due to inadequate detailing of shear reinforcement or confinement, especially at plastic hinge zone 	Damages due to the compromised continuity of the element (i.e. inadequate splice and detailing)
	Short column mechanism 	Shear-axial failure due to insufficient confinement under high axial load  Torsional cracks in circular columns due to lack of transverse reinforcement 
Joint	Failure of joint area, possibly due to absence of horizontal and/or vertical transverse reinforcement, inadequate anchorage of beam longitudinal bars, use of plain round bars in old days  	
Wall	Buckling of wall due to insufficient longitudinal reinforcement (especially at splice region) or excessive wall slenderness ratio Wall boundary zone compression crushing and buckling due to lack of detailing of shear reinforcement and confinement  	Out-of-plane damages (wall buckling, concrete crushing or spalling, etc.) due to irregular shapes, inadequate confinement  

6.2.4. Other Observed Damages

Table 6- 4: Damages observed and summarised in the refined building database for secondary structural components and nonstructural components

Secondary Structural Components and Nonstructural Components	Damages	
Stair	For a few high-rise buildings: complete or partial collapses of internal precast concrete staircase 	For many mid to high-rise buildings: minor to moderate levels of movements and damages 
Diaphragm	Floor-to-wall diaphragm: damage of connections  	Floor-to-frame diaphragm:  
Nonstructural Components	Vertical components: communication equipment, glass facades, parapets, windows, masonry infill block walls, partition walls, plasters, chimneys, gypsum boards, weatherboards, etc.   	Horizontal components: ceiling tiles, service ducts, HVAC units, water pipes, roof tiles, etc.  

Table 6- 5: Damages due to geotechnical issues

Geotechnical Issues	Damages
Liquefaction	Differential settlement of RC buildings Buildings with pile foundations least affected High-rise buildings with shallow foundation most significantly affected  
Geotechnical movements, e.g. Lateral spreading of ground	Separation/cracks at foundation, ground slab, leaning of base columns and upper level structures   

One of the most alarming results of the Canterbury earthquake sequence is that the secondary structural components and nonstructural components were subject to destructive damages, especially stairs, diaphragm, and nonstructural components stated in Table 6- 4. Therefore, high attention has been attracted regarding the retrofitting and new designing of such elements, in aiming to increase capacities to sustain the potentially high seismic demand and reduce potential damages after earthquakes.

As shown in Table 6- 5, the induced damages to buildings due to geotechnical issues are summarised. Severe liquefaction was observed in Christchurch after earthquakes, causing significant differential settlements of buildings and houses, particularly in eastern part of Christchurch. Also, the ground especially near river side or lake side was subject to severe lateral spreading, leading to the leaning of the superstructures.

Due to scope of the thesis and timeframe constraint, the assessment of secondary structural components and nonstructural components and the investigation in geotechnical issues induced damages is not included. Future researches are required.

6.3. Building Information

It is worth noting that the building information included in this section is retrieved only from the Refined Database. In other words, the building cases discussed in the thesis is only limited to the reinforced concrete buildings from the refined database.

6.3.1. Knowledge Levels and Knowledge Factors (or confidence factors)

By screening the Refined Database, it is obvious that the levels of amount or quality of building information vary for different buildings. Among the 154 buildings, only 12 of them are provided with the full detailed structural drawings and technical reports (including details of material and component properties for design), 10 of them are provided with limited structural drawing, material and component information, and for the rest, no structural drawings are available. Hence, two knowledge levels can be considered – Level 1 (no structural drawings available) and Level 2 (with some or full structural drawings). Corresponding to the different knowledge levels, different levels of assessment and analyses should be selected. For instance, for the buildings of Knowledge Level 1, ISA can be conducted directly, and if DSA is required, more information, such as structural drawings, condition reports, survey data, etc. should be collected. For the buildings of Knowledge Level 2, more sophisticated analyses (i.e. SLaMa, NSP&LPA or NDP) can be applied with the sufficient knowledge.

6.3.2. Building Typology

As discussed in Section 6.1, in the Database, four typical reinforced concrete building types are defined together with sub-types determined according to building age (i.e. 8 groups) and height (i.e. 5 groups). However, due to the scope of thesis, simper building typology system is suggested based on the study from PAGER-STR, RISK-UE (which is the existing taxonomy suited to European structures) and SYNER-G Project.

PAGER taxonomy, on the basis of existing taxonomy and tailored for worldwide structures, identifies a few main classes and some sub-classes. The taxonomy concerning the reinforced concrete structure types is shown in Table 6- 6.

RISK-UE building classification has been developed on the basis of existing taxonomy and adjusted for European structures. The RISK-UE matrix comprises 23 principal classes grouped by the structural types and materials of construction, and further sub-classes are defined according to the three different height classes (i.e. low-rise, mid-rise and high-rise). A building design code and a performance level (i.e. pre-code, low-code, moderate-code or high code) are also assigned to each of the categories. In Table 6- 7, only classes concerning reinforced concrete structures are shown.

Compared to PAGER taxonomy and RISK-UE classification, Syner-G Project includes more detailed information, as shown Table 6- 8. Syner-G taxonomy is constructed with a modular structure. Such modular system makes it possible to take account for all types of structures and makes it convenient to add more categories or sub-categories. The categories/sub-categories are defined considering concrete properties and strength, rebar properties and strength, reinforcing details, plan irregularities, vertical irregularities, cladding characteristics, floor characteristics, roof characteristics, height, and whether designed to seismic code, etc.

In addition, as presented in NZSEE 2015 Conference, an innovative, globally applicable building taxonomy was developed for the Global Earthquake Model (GEM) to consistently describe and classify buildings. As stated in the abstract of the paper, "*The Taxonomy's potential applications extend beyond seismic risk – it can be used to facilitate global collaboration on the diversity of vulnerability of the world's existing buildings*". Due to the scope of thesis, no more details regarding GEM Taxonomy are included, and the overall building taxonomy and glossary are available at http://www.nexus/globalquakemodel.org/gem-building-taxonomy/overview.

Table 6- 6: PAGER reinforced concrete structure typology

Label	Description	L (1-3)	M (4-7)	H (8+)
C1	Ductile reinforced concrete moment frame	C1L	C1M	C1H
C2	Reinforced concrete shear walls	C2L	C2M	C2H
C3	Non-ductile reinforced concrete frame with masonry infill walls	C3L	C3M	C3H
C4	Non-ductile reinforced concrete frame without masonry infill walls	C4L	C4M	C4H
C5	Steel reinforced concrete (steel members encased in reinforced concrete)	C5L	C5M	C5H
PC1	Precast concrete tilt-up walls			
PC2	Precast concrete frames with concrete shear walls	PC2L	PC2M	PC2H

Table 6- 7: RISK-UE reinforced concrete structure typology

Label	Description	L (1-2)	M (3-5)	H (6+)
RC1	RC moment frames	RC1L	RC1M	RC1H
RC2	RC shear walls	RC2L	RC2M	RC2H
RC31	Regularly infilled RC frames	RC31L	RC31M	RC31H
RC32	Irregular RC frames	RC32L	RC32M	RC32H
RC4	RC dual systems	RC4L	RC4M	RC4H
RC5	Precast concrete tilt-up walls	RC5L	RC5M	RC5H
RC6	Precast concrete frames with concrete shear walls	RC6L	RC6M	RC6H

Table 6- 8: Syner-G Project reinforced concrete structure typology

Description		Main		Sub	
FRM	**Force Resisting Mechanism**	**FRM1**		**FRM2**	
		MRF	Moment Resisting Frame	EB	Embedded beams
		W	Structural Wall	EGB	Emergent beams
		FS	Flat Slab		
		BW	Bearing Wall		
		P	Precast		
		CM	Confined Masonry		
FRMM	**FRM Material**	**FRMM1**		**FRMM2**	
		C	Concrete	RC	Reinforced concrete
		M	Masonry	URM	Unreinforced masonry
				RM	Reinforced masonry
				HSC	High strength concrete (>50Mpa)
				ASC	Average strength concrete (20-50Mpa)
				LSC	Low strength concrete (<20Mpa)
				A	Adobe
				FB	Fired brick
				HC	Hollow clay tile
				S	Stone
				HY	High yield strength reinforcing bars (>300Mpa)
				LY	Low yield strength reinforcing bars (<300Mpa)
				A/B/C	Classification of reinforcing bars based on EC2
				LM	Lime mortar
				CM	Cement mortar
				MM	Mud mortar
				SB	Smooth rebars
				NSB	Non-smooth rebars
				CMU	Concrete masonry unit
				AAC	Autoclaved aerated concrete
				H%	High % of voids
				L%	Low % of voids
				Rc	Regular cut
				Ru	Rubble
P	**Plan**	R	Regular		
		IR	Irregular		
E	**Elevation**	R	Regular geometry		
		IR	Irregular geometry		
C	**Cladding**	**C**		**CM (Cladding Characteristics)**	
		RI	Regular infill vertically	FB	Fired brick masonry
		IRI	Irregular infill vertically	H%	High % voids
		B	Bare	L%	Low % of voids
				AAC	Autoclaved aerated concrete
				PC	Precast concrete
				G	Glazing
				SL	Single layer of cladding
				DL	Double layer of cladding
				P	Open first floor (Pilotis)
				U	Open upper floor
D	**Detailing**	D	Ductile		
		ND	Non-ductile		
		WTB	With tie rods/beams		
		WoTB	Without tie rods/beams		
FS	**Floor System**	**FS**		**FSM (Floor System Material)**	
		R	Rigid	RC	Reinforced concrete
		F	Flexible	S	Steel
				T	Timber
RS	**Roof System**	**RS**		**RSM (Roof System Material)**	
		R	Rigid	T	Timber
		F	Flexible	Th	Thatch

				CMS	Corrugated metal sheet
HL	**Height Level**	**HL**		**NS (Number of stories)**	
		L	Low rise, 1-3		
		M	Mid-rise, 4-7		
		H	High-rise, 8-19		
		Ta	Tall, 20+		
CL	**Code Level**	NC	None		
		LC	Low, <0.1g		
		MC	Moderate, 0.1-0.3g		
		HC	High, >0.3g		

Table 6- 9: Building typology defined (left: Knowledge Level 1; right: Knowledge Level 2) in thesis

Main Type	Building Age	Height
Frame	Pre1970s (including 70s)	Low-rise (1-3 storey)
		Mid-rise (4-7 storey)
		High-rise (8+ storey)
	Post 1980s	Low-rise (1-3 storey)
		Mid-rise (4-7 storey)
		High-rise (8+ storey)
Shear Wall	Pre1970s (including 70s)	Low-rise (1-3 storey)
		Mid-rise (4-7 storey)
		High-rise (8+ storey)
	Post 1980s	Low-rise (1-3 storey)
		Mid-rise (4-7 storey)
		High-rise (8+ storey)

Main Type	Building Age	Height
Frame	Pre1970s (including 70s)	Low-rise (1-3 storey)
		Mid-rise (4-7 storey)
		High-rise (8+ storey)
	Post 1980s	Low-rise (1-3 storey)
		Mid-rise (4-7 storey)
		High-rise (8+ storey)
Frame-Shear Wall Dual System	Pre1970s (including 70s)	Low-rise (1-3 storey)
		Mid-rise (4-7 storey)
		High-rise (8+ storey)
	Post 1980s	Low-rise (1-3 storey)
		Mid-rise (4-7 storey)
		High-rise (8+ storey)
Frame with Masonry Infill	Pre1970s (including 70s)	Low-rise (1-3 storey)
		Mid-rise (4-7 storey)
		High-rise (8+ storey)
	Post 1980s	Low-rise (1-3 storey)
		Mid-rise (4-7 storey)
		High-rise (8+ storey)
Shear Wall	Pre1970s (including70s)	Low-rise (1-3 storey)
		Mid-rise (4-7 storey)
		High-rise (8+ storey)
	Post 1980s	Low-rise (1-3 storey)
		Mid-rise (4-7 storey)
		High-rise (8+ storey)

The suggested building typology applied in thesis is shown in Table 6- 9. The table on the left side gives the typology defined corresponding to Knowledge Level 1. Only two main structural types, frames and shear walls, are defined due to lack of information. The sub-types are defined according to two building age classes (i.e. pre 1970s including 1970s and post 1980s) and three building height classes (i.e. low-rise, mid-rise and high-rise). The table on the right side gives a more refined building classification corresponding to Knowledge Level 2. Four main structural types, bare frame, frame-wall dual system, frame with masonry infill and shear wall, are defined, and the sub-types are defined according to the same building age and height classes as discussed in previous. It is worth noting that the clarification of building typology is fundamental in assessing seismic response of buildings, and is vital in developing global structural models explained in Section 5.5.3. Also, the refinement of building classification can be accomplished if more building information is obtained.

6.3.3. Material Properties or Strengths

6.3.3.1. Concrete

6.3.3.1.1. Concrete compressive strength of the buildings in the Refined Database

Knowledge Level 1 buildings:

For most of the buildings of Knowledge Level 1, there is a lack of information regarding material and component properties, thus, assumptions should be made referring to the design standard at the time the building was designed or constructed. Alternatively, assumptions can be made based on the information of the similar building classes of Knowledge Level 2. With these assumptions made, uncertainties exist, and the effect on the assessment results due to the variation of material strengths is discussed in Chapter 7 and Chapter 9.

Knowledge level 2 buildings:

The nominal compressive strength of concrete found in the structural drawings or technical reports of the 22 buildings of Knowledge Level 2 are summarised in Table 6- 10. It has been found that for the pre-70s (including 1970s) buildings, the concrete strength is within a range of [17Mpa, 35Mpa], and [25Mpa, 35Mpa] for the post-80s buildings. There is a lack of information of the older concrete and the concrete properties in 1990s.

Table 6- 10: Concrete nominal strength of the 22 buildings of Knowledge Level 2

Year	Concrete strength specified in drawings or report (Nominal material strength)
1960-1969	5000psi (≈34.5Mpa) 4000psi (≈27.5Mpa) 3000psi (≈20.5Mpa) 2500psi (≈17.5Mpa)
1970-1979	3500psi (≈24Mpa) 3000psi (≈20.5Mpa)
1980-1989	25Mpa~35Mpa
1990-1999	No data found
2000-2009	30Mpa

6.3.3.1.2. Concrete properties and strengths specified in New Zealand design history

Table 6- 11, referred to the work of Amir Malek (UC PhD), provides a summary of concrete compressive strength in New Zealand design history. More detailed information, such as modulus of rupture, direct tensile strength, elastic modulus, Poisson ratio, coefficient of thermal expansion, shrinkage, creep, stress-strain curves, and applicable density range, is shown in Appendix A7, provided by Amir.

Table 6- 11: Specified concrete compressive strength from New Zealand standards

Pre 60s	1960-1969	1970-1979	1980-1989	1990-1999	2000-2009
Not specified	**NZS3101:1970** 17.2Mpa, 20.7Mpa, 27.6Mpa, 34.5Mpa	**Not specified**	**NZS3101:1982** 20Mpa$<f'_c<$55Mpa	**NZS3101:1995** 17.5Mpa$<f'_c<$100 Mpa	**NZS3101:2006** 25Mpa$\leq f'_c<$100Mpa 25 Mpa$\leq f'_c<$75Mpa (for ductile elements and elements of limited ductility)

6.3.3.2. Reinforcing Steel

6.3.3.2.1. Reinforcing steel tensile yield strength of the buildings in the Refined Database

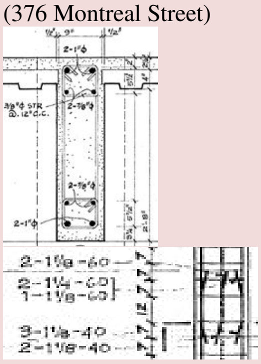
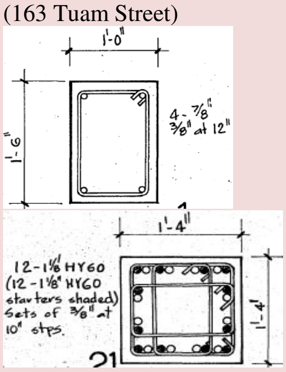
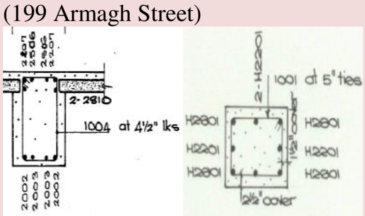
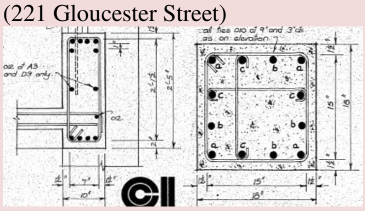
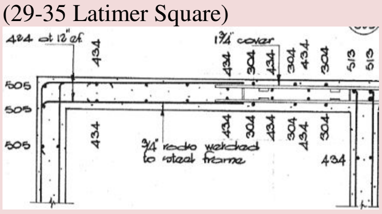
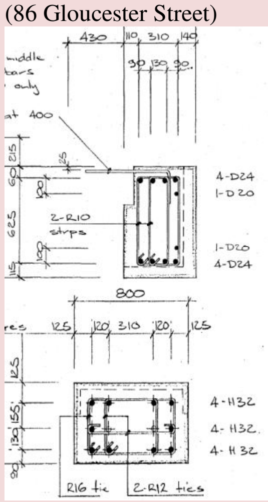
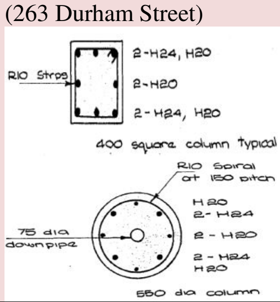
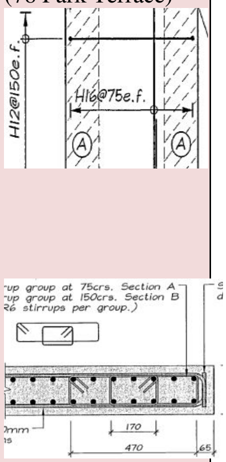
Knowledge Level 1 buildings:

See Section 6.3.3.1.1. Assumptions should be made referring to the design standard at the time the building was designed or constructed, or should be based on the information of the similar building classes of Knowledge Level 2.

Knowledge Level 2 buildings:

Table 6- 12 summarises the reinforcing steel tensile yield strength of the 22 reinforced concrete buildings of Knowledge Level 2 in the Refined Database. Various nomenclatures have been found in the structural drawings, and the detailed information is presented in Appendix A7. It is worth noting that no sufficient data concerning reinforcing steel properties was found for buildings designed or constructed pre1960s and in the period 1990-1999. Table 6- 13 provides a summary of reinforcing steel yield strength specified in New Zealand design history. It has been confirmed that the strengths specified in the structural drawings or reports are consistent with the standards.

Table 6- 12: Reinforcing steel tensile yield strength of the 22 buildings of Knowledge Level 2 with component section profiles (Note that the Table does not show reinforcing steel details for all the 22 buildings)

1960-1969	1970-1979	1980-1989	2000-2009
40000psi(276Mpa) **60000psi(414Mpa)** E.g. Strategy House (376 Montreal Street)  **HY60 Grade (414Mpa)** E.g. Old CHC City Council Building (163 Tuam Street) 	**D/R: 275Mpa** **H: 380Mpa** E.g. Trade Union Centre (199 Armagh Street)  **40ksi.(276Mpa)** E.g. Securities House (221 Gloucester Street)  **275Mpa (NZSS1693)** **HY60 Grade (NZSS1879)** E.g.AMI Building (29-35 Latimer Square) 	**D/R: 275Mpa** **H: 380Mpa** E.g. Brannigans (86 Gloucester Street)  E.g. Amuri Courts (263 Durham Street) 	**D/R: 300Mpa** **H: 430Mpa** E.g. Terrace on the Park (78 Park Terrace) 

6.3.3.2.2. Reinforcing steel tensile yield strength specified in New Zealand design history

Table 6- 13, referred to the work of Giuseppe Loporcaro (UC PhD), provides a summary of reinforcing steel yield strength in New Zealand design history, and can be used as a good reference during assessing material strength, especially under the circumstance of lacking sufficient building information.

Table 6- 13: Reinforcing steel standard development in New Zealand and the specified tensile yield strength

Pre 60s	1960-1969	1970-1979	1980-1989	2000-2009
BS165-1929 Not specified **NZS197–1949 (BS785–1938)** Mild steel: Not specified Medium tensile: 19.5tsi(≈270 Mpa) 18.5tsi(≈255 Mpa) 17.5tsi(≈241 Mpa) 16.5tsi(≈227 Mpa) 16.5tsi(≈227 Mpa) High tensile: 23.0tsi(≈317 Mpa) 22.0tsi(≈303 Mpa) 21.0tsi(≈290 Mpa) 20.0tsi(≈275 Mpa) 19.0tsi(≈262 Mpa)	**NZSS1693:1962** 33000 psi (≈ 227 Mpa) **NZSS1879:1964** HY60 Deformed 60000 psi (≈ 415 Mpa) **NZSS1693 Amendment 1 (1968)** 40000 psi (≈ 276 Mpa)	**NZ 3432P:1972** 40000 psi (≈ 276 Mpa) **NZ 3402P:1973** Grade 275 (275Mpa) Grade 380 380 Mpa	**NZS 3402:1989** Grade 300 (300-355 Mpa) Grade 430 (430-500 Mpa)	**AS/NZS 4671:2001** Grade 300 (300-355 Mpa) Grade 500 (500-600 Mpa) **AS/NZS 4671 Amendment 1(2003)** Grade 300 (300-380 Mpa) Grade 500 (500-600 Mpa)

6.3.4. Component Properties and Strengths

6.3.4.1. Beams

6.3.4.1.1. Beam properties found for buildings in the Refined Database

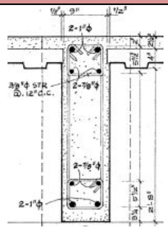
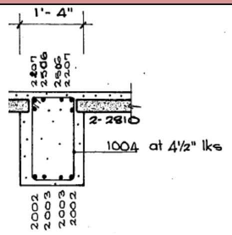
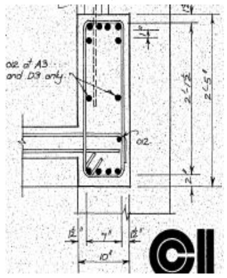
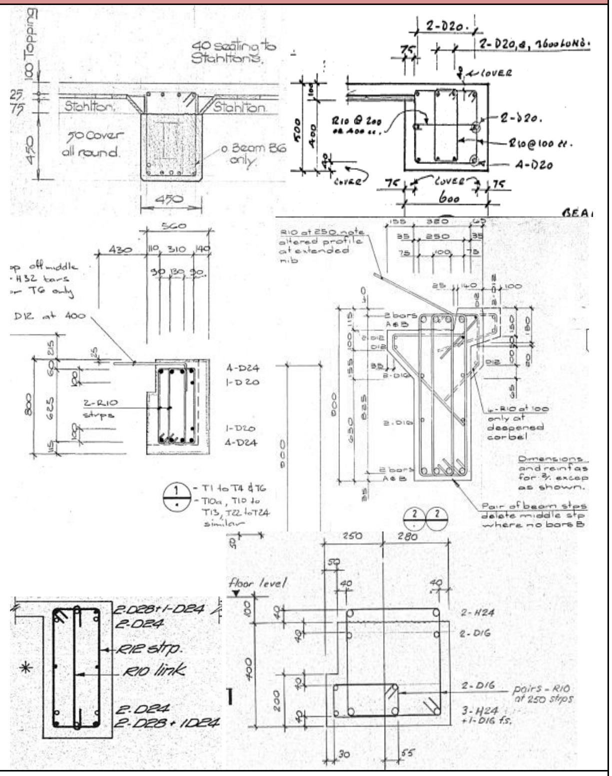
Knowledge level 1 buildings:

As mentioned in the previous section, due to the lack of sufficient information of materials and components, only a rough estimation of the beam strength can be obtained. Assumptions can be made based on the limited building information available and design standards applied when the buildings were designed or constructed. The strengths obtained should have larger varying ranges than the strengths obtained for knowledge level 2 buildings, and the effect of such variation of component strengths is discussed in Chapter 7 and Chapter 9.

Knowledge level 2 buildings:

Beam properties of 9 frame-type reinforced concrete structures of Knowledge Level 2, including the geometry and reinforcing details, are summarised in Table 6- 14.

Table 6- 14: Beam properties specified in structural drawings or technical reports

Properties	1960-1969	1970-1979	1980-1989
Section sketches		 	
Beam width (mm)	254	406.4 203.2 (254 LV1)	450 600 560/400PC,560/700IS 320 450(FR2/5/C/E), 350(FRC/E LV7~10), 650 (FR3/4) 480 (FR A/H), 400 (FR C/F)
Beam height (mm)	812.8 (977.9 DIR)	609.6 736.6	600(405 LV3~5 EX) 500 800PC,1000/ 650/950/500/800IS 900 900 (FR 2/5/C/E), 600 (FR 3/4) 400
Beam length (mm)	9792 (4588 DIR)	9144(4876.8DIR) 6794.5 (5067.3 DIR)	2130, 4725, 6962, 3100, 8405, 5840, 3836 5800, 5212, 5086, 6500, 7384, 7327 3430/3550 PC, 4500/3250 IS 5410/ 5495/ 5900/ 6050/ 6000/ 4888/ 4988 9900/ 4310/ 8200/ 8000/ 5380 5450/ 5350/ 5200/ 5539/ 5347/ 7000
Beam longitudinal reinforcing bars	D32, 28, 25, 22, 19	D28, 25, 22 D24, 20, 16	D24, 20 D24, 20, 16 D32, 28, 24, 20, 16 D32, 28, 24, 20, 16, 12 D28, 24, 20, 16 H32, 28, 24, 20, 16 D16
Beam longitudinal reinforcing ratio	1.112%~2.308%	1.046%~2.004% 0.907%~2.331%	0.903%~2.025% 0.628%~1.768% 0.404%~1.268% 1.213%~2.592% 1.101%~1.983% 1.047%~3.676%
Beam transverse reinforcing bars	R10@304.8 R10@457	R10@114.3 (PPHZ) R10@228.6(NPPHZ) R10@177.8	R10@50 (PPHZ), @250 R10@100 (PPHZ), @200,400 R10@100(IN),250(EX),150(D,EX),140(D, upIN,low EX) R10@80, 120 R10@115/130/135/160/200/300,12@140/250 R10@ 100/160/250/

6.3.4.1.2. Beam properties and strengths specified in New Zealand design history

The information concerning beam properties and the determination of strengths specified in New Zealand design history can provide good references during assessing reinforced concrete beam elements. The information can be in a table-format (similar format to the table in Appendix A8, discussed in Section 6.3.4.2.2), including with section sketches, geometry descriptions, parameters, numbers and formulae from New Zealand design standards. This requires future research and investigation.

6.3.4.2. Columns

6.3.4.2.1. Column properties found for buildings in the Refined Database

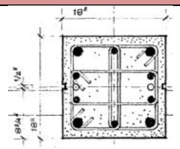
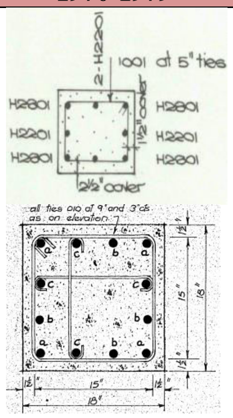
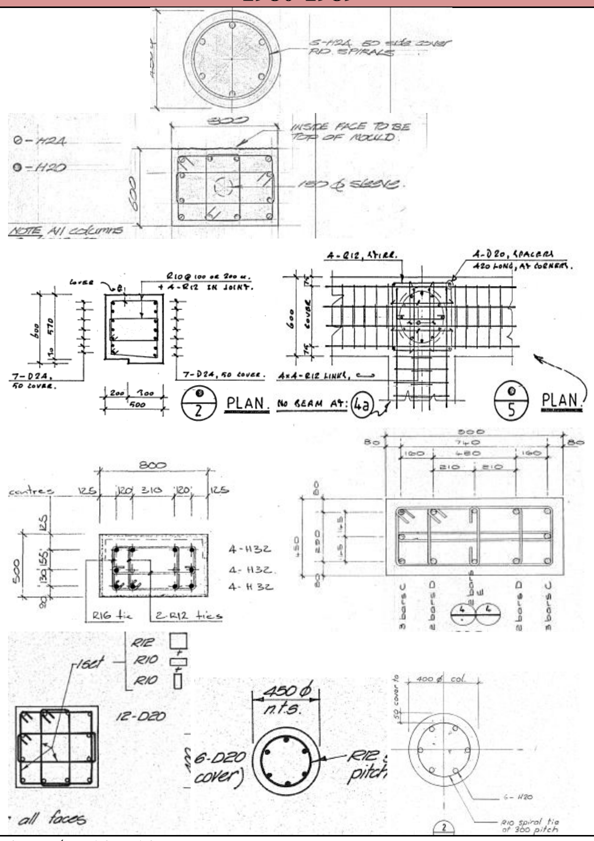
Knowledge level 1 buildings:

See Section 6.3.4.1.1.

Knowledge level 2 buildings:

Column properties of 9 frame-type reinforced concrete structures of Knowledge Level 2, including the geometry and reinforcing details, are summarised in Table 6- 15.

Table 6- 15: Column properties specified in structural drawings or technical reports

Properties	1960-1969	1970-1979	1980-1989
Section sketches			
Column width (mm)	457.2 (508 FRB)	457.2 457.2	600 (Ø=356 (456 IN)) 500 (Ø=450 (600*600)) 500PC,500/625 IS 450 PC, 550 IS 500(FR2/5/C/E),900(FR3/4 EX),600 (FR3/4 IN) Ø=400 (FR A/H), 600 (FR C/F)

Column height (mm)	457.2(635JO,558.8FRB)	508(1524FRC,457.2TLV) 457.2	800 (Ø=356 (456 IN)) 600 (Ø=450 (600*600)) 800PC,800/600 IS 900 900EX,1000IN(FR2/5),800EX,900IN (FRC/E),500(FR2/4 EX),600(FR3/4 IN) Ø=400 (FR A/H), 600 (FR C/F)
Column length (mm)	3772	3657.6 (2743.2 LV1) 3048	3420 (3960 LV1, 3000 TLV) 3300 3400 (3600 LV1, 2800 LV2) 3200 (4000 GLV) 3275(3600 LV1) 3400
Column longitudinal reinforcing bars	60/40 – 32, 28, 25	H32, 28, 22 (16, 13) D28	H24, 20 H24 H32, D32, 28, 24, 20 H32, 28, 24, 20, 16 D32, 28, 24, 20 H32, 24, 20
Column longitudinal reinforcing ratio	1.109%~4.583%	0.931%~2.357% 1.178%~3.535%	0.901%~2.727% 1.810%~2.413% 0.905%~2.721% 1.129%~1.955% 0.860%~2.083% 1.280%~2.276%
Column transverse reinforcing bars	R10@304.8, 177.8, 279.4, 152.4, 254 R16@80.43	R10@127 (PPHZ), 304.8 R10@228.6 (76.2 at SP)	R10@50 at JO, @100 at SP, @200,300 R10@90,100(PPHZ), @180,200 EX: LV1~3: 2H20@300 & 3R10@100 LV3~8: 2D16@200 & 3R10@100 LV8~11:2R10@100 IN: LV1~2: 3R20 or (R16+2R12) @100 LV2~8: (R16+2R12) @100 PHZ, (R16+2R12) @200other LV8~11: 2R10@250 R16@ 75, R12@ 90 R10@100,12@65/100/130/120,16@100 R10@ 300

6.3.4.2.2. Column properties and determination of strengths specified in New Zealand design history

The information concerning column properties and the determination of strengths specified in New Zealand design history can provide good references during assessing reinforced concrete beam elements. The table shown in Appendix A8, retrieved from Arsalan Niroomandi (UC PhD)'s research on gravity columns and super columns, provides a completed summary of column properties and strengths applied along New Zealand design history. However, in order to improve the current New Zealand assessment guidelines by including these information, future research and investigation is required.

6.3.4.3. Joints

6.3.4.3.1. Joint properties found for buildings in the Refined Database

Knowledge level 1 buildings:

See Section 6.3.4.1.1.

Knowledge level 2 buildings:

Table 6- 16: Joint properties specified in structural drawings or technical reports

Properties	1960-1969	1970-1979	1980-1989	1990-1999	2000-2009
Section Sketches					
Beam width					
Beam height					
Column width					
Column heigt					
Beam longitudinal reinforcing					
Beam transverse reinforcing					
Column longitudinal reinforing					
Column transverse reinforcing					
Joint reinforcing					
Splice at joint					
Etc.					

This Table should be completed by future researches and investigations.

6.3.4.3.2. Joint properties and strengths specified in New Zealand design history

This requires future research and investigation, for instance, a summary of joint properties and strengths applied along New Zealand design history can be obtained from Alberto Cuevas Ramirez (UC PhD)'s research on joint residual strength.

6.3.4.4. Walls

The wall properties found for buildings in the Refined Database, wall properties and strengths specified in New Zealand design history should be summarised. Due to timeframe constraint and scope of thesis, the summaries are not completed.

6.3.4.4.1. Wall properties found for buildings in the Refined Database

Table 6- 17: Wall properties specified in structural drawings or technical reports for 3 wall-type reinforced concrete buildings of Knowledge Level 2

Properties	1960-1969	1970-1979	1980-1989	1990-1999	2000-2009
Section Sketches					
Wall thickness					
Wall length					
Wall height					
Aspect ratio					
Longitudinal reinforcing bars					
Longitudinal reinforcing ratio					
Transverse reinforcing bars					
Transverse reinforcing ratio					
Boundary elements					
Additional confinement					
Etc.					

This Table should be completed by future researches and investigations.

6.3.4.4.2. Wall properties and determination of strengths specified in New Zealand design history

This requires future research and investigation, for instance, a summary of wall properties and strengths applied along New Zealand design history can be obtained from Farhad Dashti (UC PhD)'s research on shear walls. In addition, a comparison between the design standards NZS3101: 1995 and NZS3101: 2006 is provided in Appendix A3.

6.4. IEP Results

Before the commencement of more detailed assessment of Christchurch CBD building study cases, Initial Evaluation Procedure was carried out on the 154 buildings in the Refined Building Database. The conclusion drawn from IEP is shown in the following. IEP results, together with the determination of potential structural deficiencies (i.e. assigned values for factors) are shown in Appendix A13.

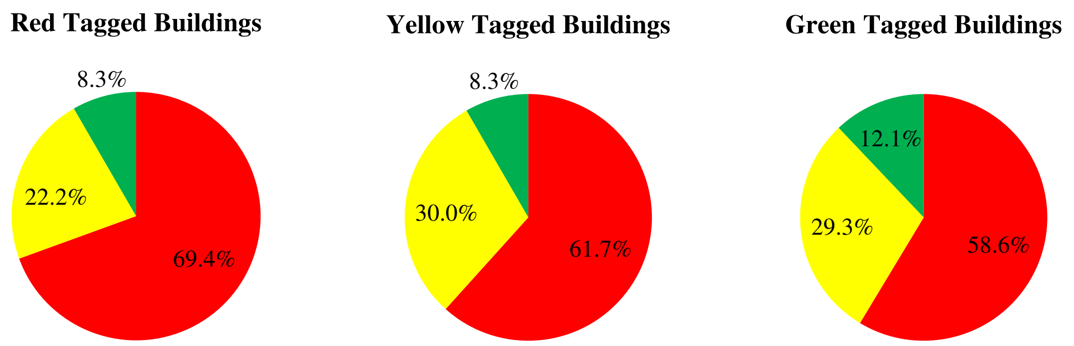


Figure 6- 12: IEP results of the red tagged, yellow tagged and green tagged buildings in the Refined Database

- Among the 36 red tagged buildings in the Refined Database, 69.4% of them were assessed to be in the category of "%NBS < 34%", 22.2% were in "34% ≤ %NBS < 67%", and 8.3% were in "%NBS ≥ 67%".
- Among the 60 yellow tagged buildings in the Refined Database, 61.7% of them were assessed to be in the category of "%NBS< 34%", 30.0% were in "34% ≤ %NBS < 67%", and 8.3% were in "%NBS ≥ 67%".
- Among the 58 green tagged buildings in the Refined Database, 58.6% of them were assessed to be in the category of "%NBS< 34%", 29.3% were in "34% ≤ %NBS < 67%", and 12.1% were in "%NBS ≥ 67%".
- From the statistical results shown in Figure 6- 1, IEP can only provide preliminary determination of potential structural deficiencies, and the %NBS values can be either conservative or vice versa due to the lack of knowledge.
- For buildings of Knowledge Level 1, before carrying out more detailed assessment, building information, such as structural drawings, construction documents, material test reports, and so on, should be collected. For buildings of Knowledge Level 2, with the building information at the hand, more detailed assessment should be carried out. In Chapter 7, building case studies from the Knowledge Level 2 building category were selected to undergo detailed seismic assessment procedures. IEP results of the selected building case studies are shown in Chapter 7 and Appendix A13.

CHAPTER 7 Case Study Building – Securities House and Alternative Case Study Buildings (with details shown in Appendix A14)

7.1. Building Information

7.1.1. Building Brief Descriptions

Figure 7- 1: Photo of the Building

Table 7- 1: Brief information of Building No.21 – Securities House

Information	Description
Building Name	Securities House (Building No.21)
Building Location	221 Gloucester Street
Number of Storey Above Ground	8
Age	1974 (built year) (1970-1979)
Structural Type	RC Frames with Walls
Building Typology	Pre-70s (including 70s) RC Mid-High-rise Frame with Shear Walls
Occupancy Type	Commercial office

As shown in Table 7- 1, Building No.21 – Securities House, located at 221 Gloucester Street, is an 8-storey reinforced concrete building which was used as commercial office before 2011 Canterbury Earthquake. The building was built in 1974, possessing one reinforced concrete perimeter frame in the longitudinal direction (i.e. Frame 1), reinforced concrete perimeter frame systems in the transverse direction (i.e. Frame A and Frame D), and C-shaped reinforced concrete structural wall along with an inner L-shaped singly reinforced wall on the east side of the building, as shown in the plan view of the building (Figure 7- 2).

The building has a regular shape, but unbalance resisting system in terms of strength (stiffness) exists due to the location of the staircase core on east side with no counterpart on the west facade.

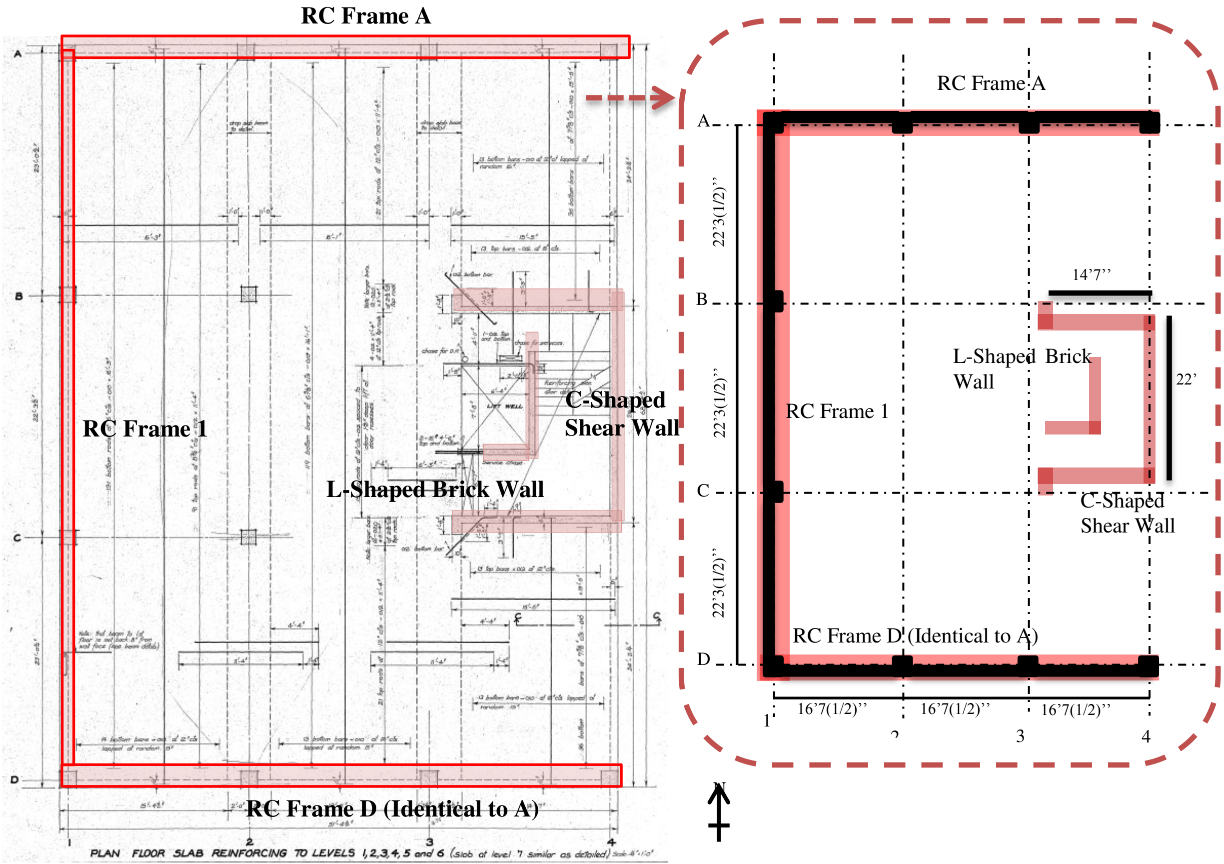


Figure 7- 2: Ground floor plan view of Building No.21 Securities House with illustration of structural systems

7.1.2. Structural Systems

Table 7- 2: Summary of critical/principle structural systems

Structural Systems or Components	Type	Load Path
Frame 1	Seismic-load-dominated RC frame	Resist lateral load from longitudinal direction Resist gravity load
Frame A	Seismic-load-dominated RC frame	Resist lateral load from transverse direction Resist gravity load
Frame D	SAME as Frame A	SAME as Frame A
C-shaped Shear Wall	Seismic-/Gravity-load-dominated RC structural core wall	Resist gravity load from stairs, slabs or building equipment Resist lateral load from both longitudinal and transverse directions
L-shaped Wall	Gravity-load-dominated wall	Resist gravity load from stairs or building equipment
Interior Frames	Gravity-load-dominated steel frames	Resist gravity load
Foundation System	Single footings for columns and deep beams for walls	Footings and deep beams are attached to the ground using piles. Footings are attached to each other using slender foundation beams.

NOTE: "Gravity load" in the table is a general term describing load from vertical direction, including live load, etc.

Gravity loads in the central area are resisted by columns with capitals joint continuously to the cast in-situ floor slabs. The perimeter frames not only take proportions of gravity loading, but also resist lateral forces, for instance, Frame 1 resisting the lateral force coming along the longitudinal direction and Frame A and D resisting the lateral force coming along the transverse direction. The C-shaped reinforced concrete wall can resist lateral loading from both directions, and it is worth noting that the performance of the corner parts of the walls is complicated to assess due to bi-directional loading. The interior L-shaped wall, made of concrete bricks with a simple mesh inside two brick layer and singly reinforced, works as a support for stairs or other building equipment. It has also been discovered that a number of gravity-dominated steel interior frames exist in the building. The foundation system is formed by single footings for columns and foundation deep beams for walls, attached to the ground using piles. Footings are attached to each other using slender foundation beams.

Plan and elevation views of the critical structural systems are shown in Figure 7- 3. Figure 7- 4 shows an elevation sketch of one of the critical structural system Frame 1.

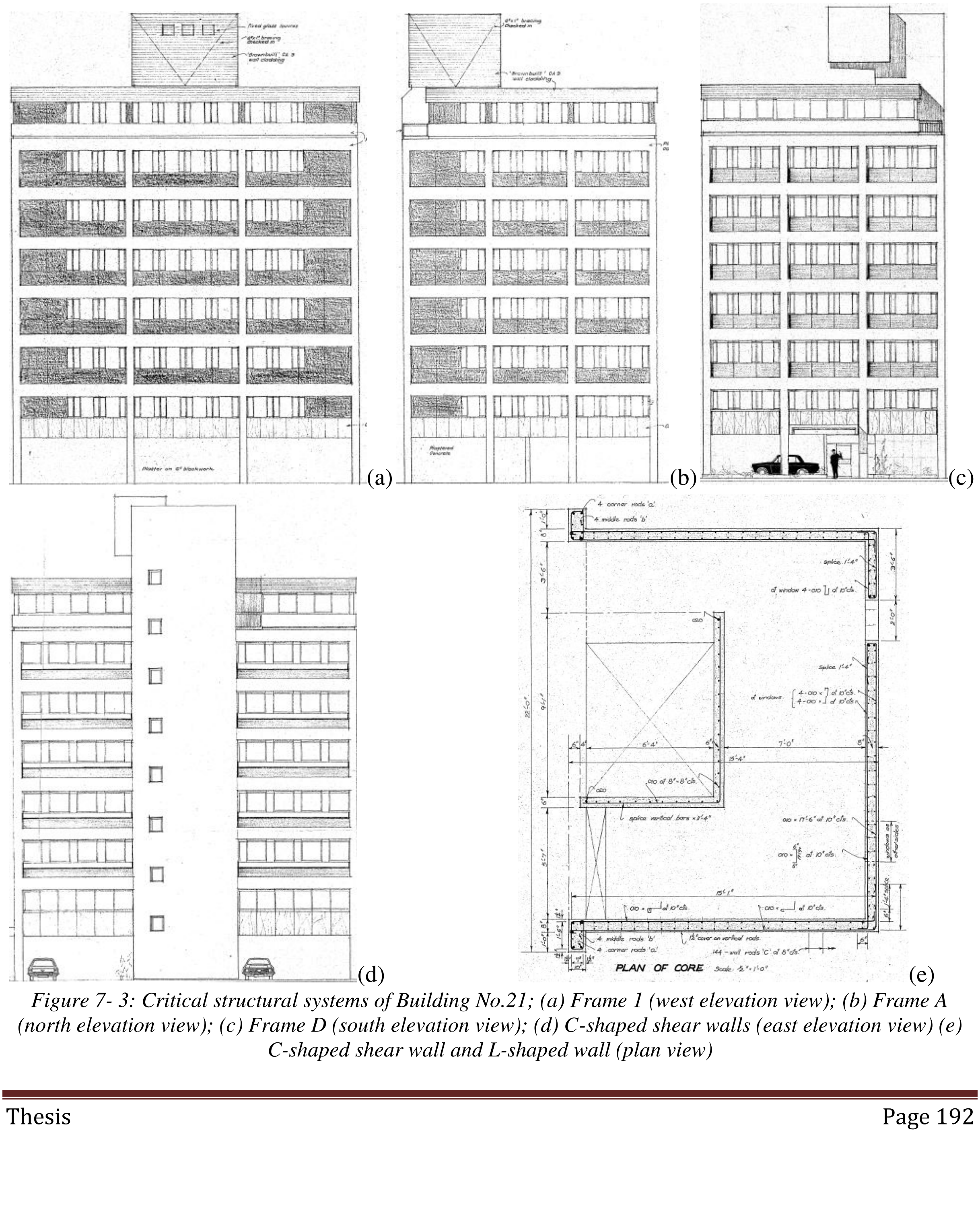


Figure 7- 3: Critical structural systems of Building No.21; (a) Frame 1 (west elevation view); (b) Frame A (north elevation view); (c) Frame D (south elevation view); (d) C-shaped shear walls (east elevation view) (e) C-shaped shear wall and L-shaped wall (plan view)

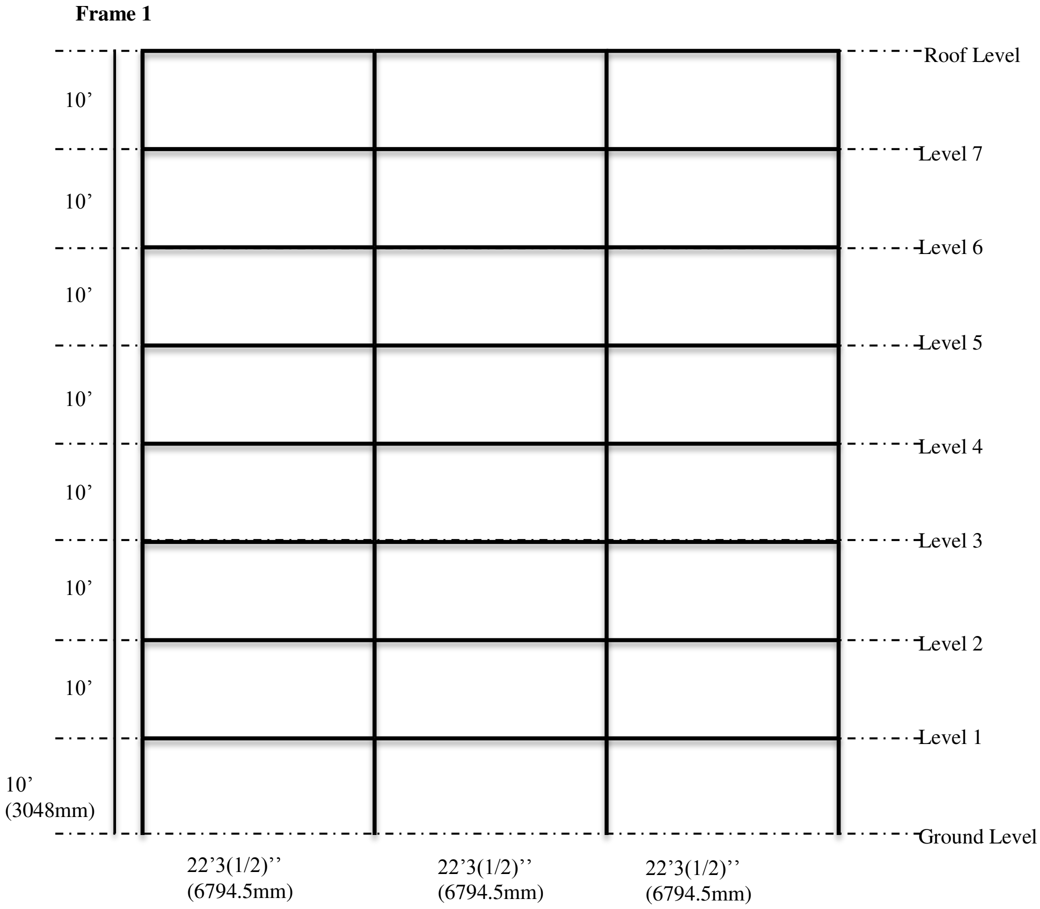


Figure 7- 4: Elevation view of Frame 1

Table 7- 3: Beam section profiles in Frame 1

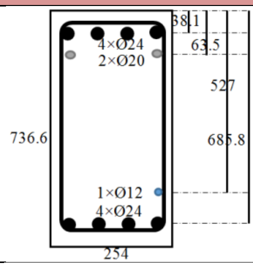
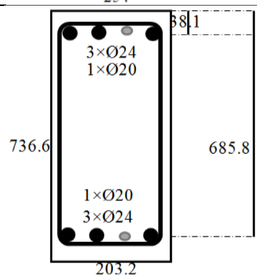
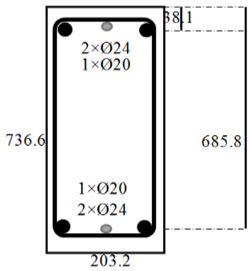
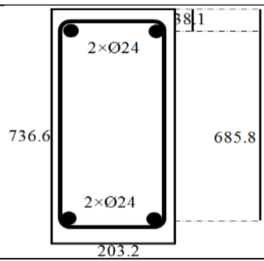
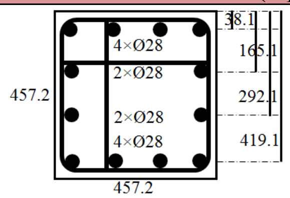
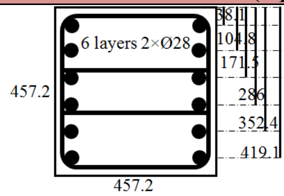
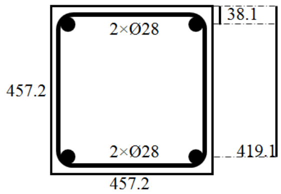
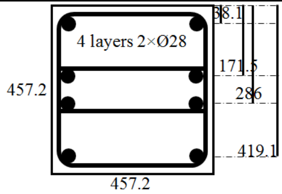
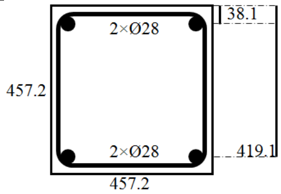
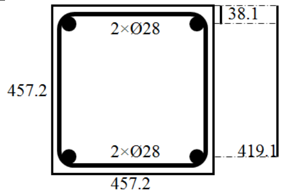
Level	Exterior Beam Sections	Interior Beam Sections
Level 1		SAME with exterior
Level 2,3,4		
Level 5,6,7,R		SAME with exterior

Table 7- 4: Column section profiles in Frame 1

Level	Exterior Column Sections (A_1, D_1)	Interior Column Sections (B_1, C_1)
Ground level to Level 4 above		
Level 4 above to Level 5 above		
Level 5 above to Roof		

In Figure 7- 4, the elevation view of Frame 1 is shown. Table 7- 3 and Table 7- 4 summarise the section profiles for beams and columns in Frame 1. The computation of component strengths is shown in Section 7.3.

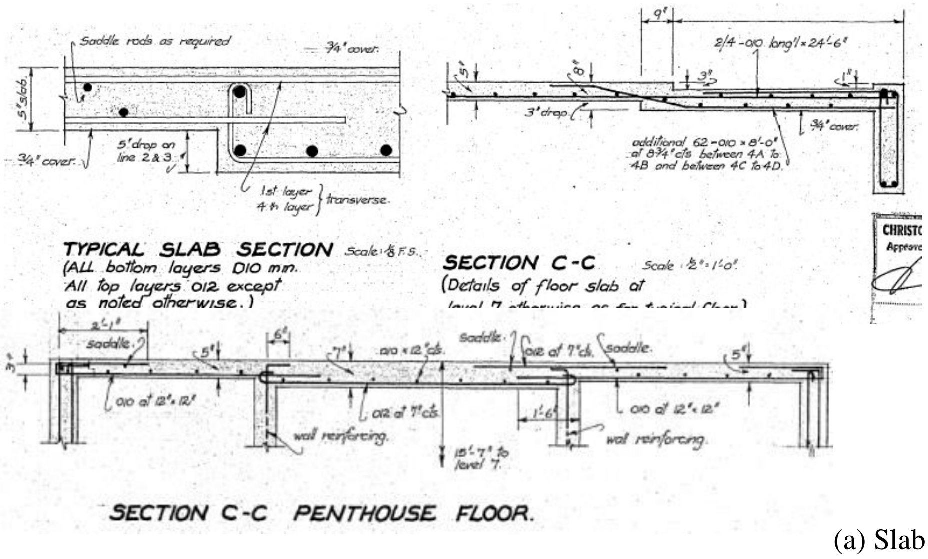
7.1.3. Secondary Structural Components and Non-structural Components

Table 7- 5 provides a summary of secondary structural components and nonstructural components. In Figure 7- 5, section details (from structural drawings) of floor slabs, stairs, brick wall and cladding system are shown.

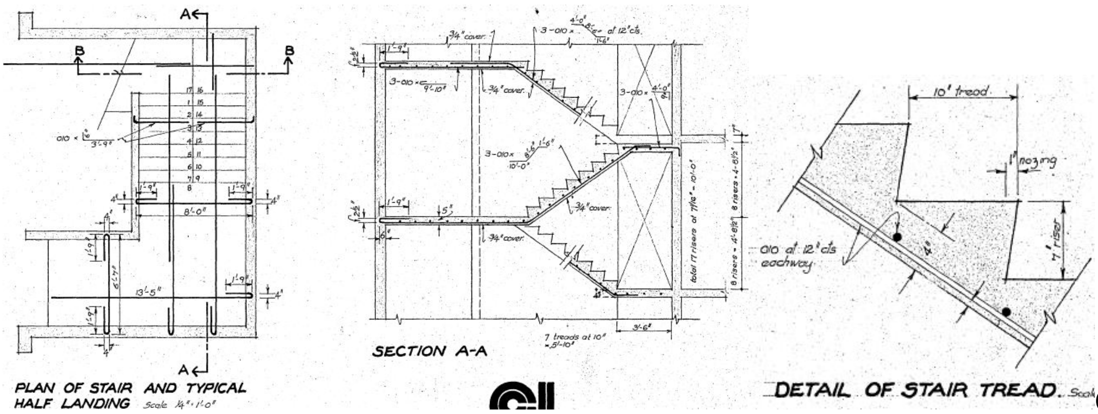
It is worth recognising that in a seismic event, the behaviour of the secondary structural components and non-structural components may significantly influence the overall response of the structure, and the cost to repair the damages caused by non-structural components is sometimes evaluated as much greater than the cost to repair the damages to the structural components. Therefore, assessment of the less critical structural components and non-structural components is necessary, and current guidelines may need to include detailed instruction to assess such components, as discussed in Section 5.1.

Table 7- 5: Summary of secondary structural components and non-structural components

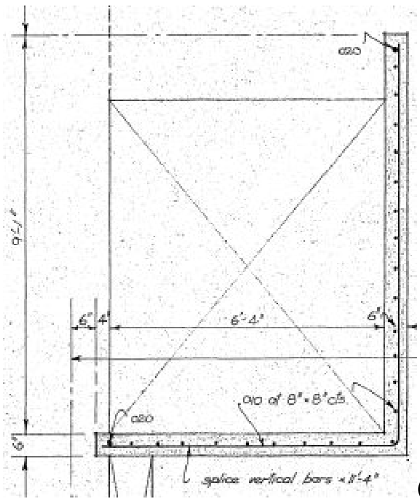
Secondary structural and non-structural components	Type	Description
Floor System	Cast-in-situ RC slab	Slabs of 10 in. (i.e. 254mm) along line 2 and 3, and 5 in. (i.e. 127mm) else where
Stairs	Two-way reinforced stair units/tread	Properly seated or connected. Stairs are jointed at each floor level by cast in situ floor slabs.
Masonry/Brick Wall (the L-shaped Wall)	Gravity load-dominated wall	Interior brick wall with simple mesh and single layer of reinforcement
Claddings/Windows	Glass	Glassing in form of stiff still windows is found to be attached rigidly to the structure



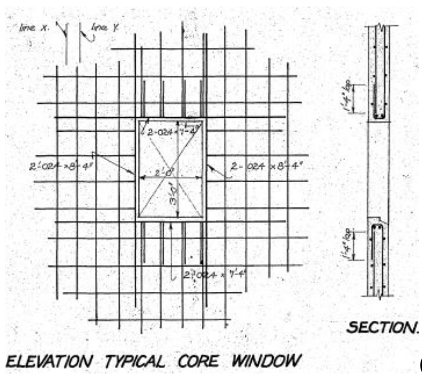
(a) Slab



(b) Stairs



(c)L-shaped wall



(d) Windows

Figure 7- 5: Secondary structural components and non-structural components; (a) Typical section profile of cast-in-situ RC slabs; (b) Typical section profile of reinforced stair tread; (c) Plan view of brick wall; (d) Elevation view of windows

7.1.4. Observed Damages

Only outside inspection (Level 1 and Level 2 Evaluation) was done, and the observed damage information is shown in the following tables.

Table 7- 6: General damage information of Building No.21 – Securities House

Inspection	General damage information
Tagging	Red
Building Damage Ratio	11-30%
Usability Rate	R2: Severe damage demolition likely
Demolition Info.	NOT on Demolished Building List (Up to 13/02/2014) (CERA)

Table 7- 7: Summary of observed structural damages

Structural Damage	Damage Description
Beams	Plastic hinge formations in external beam ends in all storeys except top floor
Columns	Severe short column damage on 1st floor
Joints	Joint hinging in internal joints in all storeys except top floor
Foundation	Minor/ None
Roof Floor	Minor/ None

Table 7- 8: Summary of observed non-structural damages

Non-structural Damage	Damage Description
Masonry Walls	Severe diagonal cracking in masonry wall on ground floor
Stairs	Minor/ None
Claddings/Windows	Severe windows damage/hazard

Table 7- 9: Summary of site hazard and geotechnical damages

Site Hazard or Geotechnical Damage	Hazard or Damage Description
Hazard of Collapse	Minor/ None
Hazard of Leaning	Moderate
Hazard to Neighbour Buildings	Moderate
Overhead Hazard	Moderate
Settlement or Slip	Minor/ None
Slope Failure	Minor/ None
Ground Movement	Minor/ None
Liquefaction	Mino/ None

The following photos show the damages observed after earthquakes.

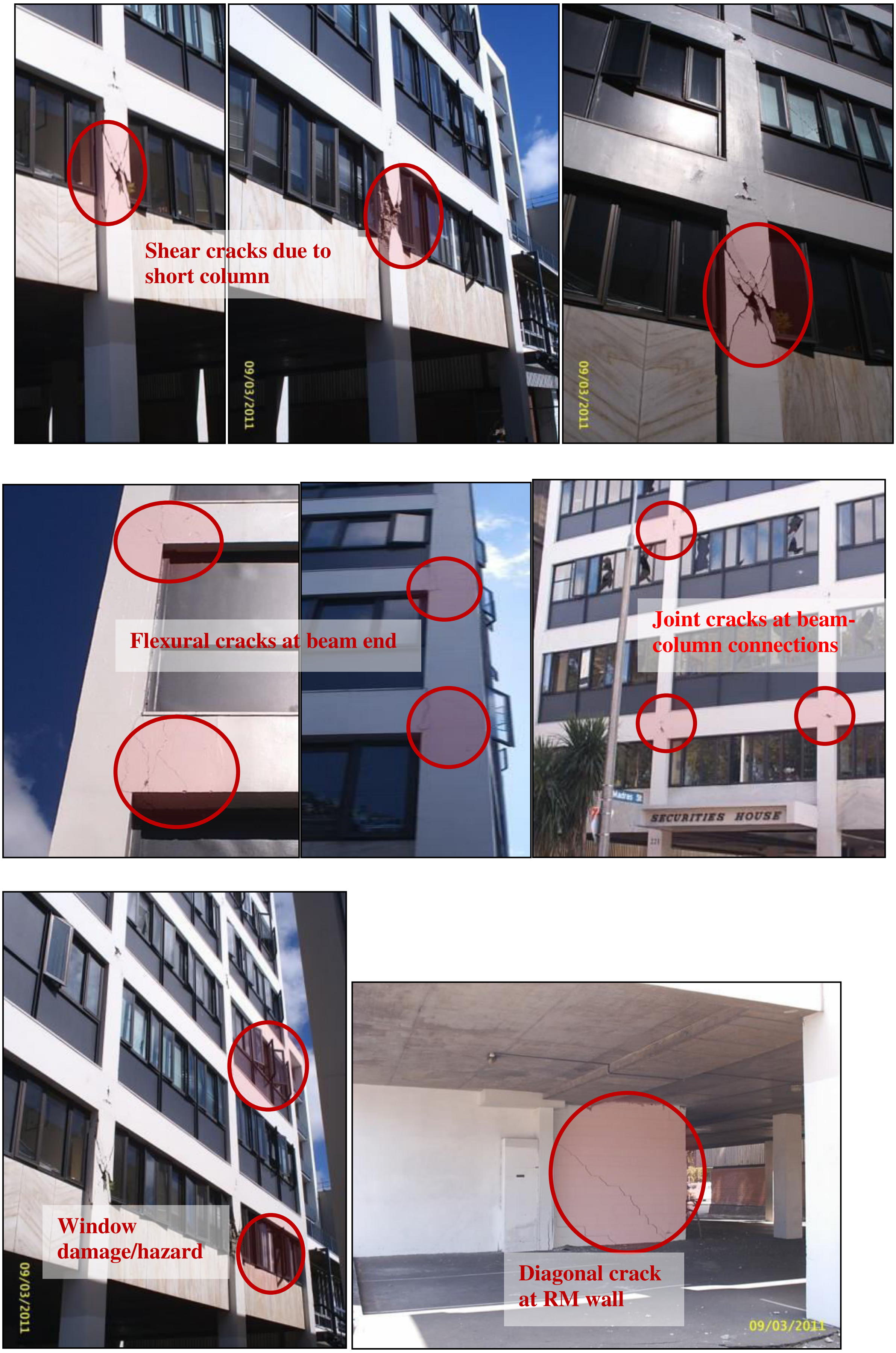


Figure 7- 6: Photos showing the observed damages to Building No.21

7.2. Initial Seismic Assessment of Building No.21

7.2.1. Preliminary Consideration on Seismic Vulnerability and Identify Critical Structural Weakness or Deficiencies

After studying the structural drawings, some issues which can be the cause of critical structural weaknesses are listed as following.

- Depth of beam sections of all levels, reading as 736.6mm, is larger than the depth of columns, 457.2mm. It is possible that the deep beams have higher flexural strength than the columns, leading to weak-columns-strong-beam mechanism based on Capacity Design Theory. It is worth noting that the building was built during mid-1970s when Capacity Design had not been introduced in design or construction yet, i.e. lack of capacity design.
- Potential inadequate reinforcement detailing has been discovered according to the drawings
  a. The amount of column longitudinal reinforcement rapidly decreases from level 4 and level 5. The reinforcement ratio of the upper levels (4~roof) is less than half of that of the lower levels (ground ~3). This may trigger soft-storey mechanism on the upper levels, causing partial or total structural failure.
  b. The L-shaped wall is only singly reinforced, but this probably will not be critical in predicting structural damage.
  c. Transverse reinforcements of beams, columns and walls are of the minimum amount. Most members only have single steel leg or hoop at large spacing distance, unable to provide proper confinement nor to prevent buckling of the longitudinal reinforcement
- C-shaped and L-shaped walls have intrinsic eccentricities (irregular shapes), and could be partially damaged under torsional effect.
- Even though the building has a regular shape, irregularity (stiffness and mass) exists due to the core wall systems with heavy stair and service loading locating on the east side of the building.
- As mentioned previously, the rapid reduction of longitudinal reinforcement in columns lead to vertical irregularity.

It was assumed that some essential elements, such as foundations, piles-caps, beams connecting core walls and frames, etc. are secured, and will not be evaluated in the following assessment procedures.

7.2.2. IEP

Table 7- 10 shows the key assumptions made during the assessment of Building No.21, referring also to the attached Appendix A13.

As shown in Table 7- 11, the initial seismic assessment of Building No.21 indicates that it can achieve 20%NBS in both the longitudinal and transverse direction, identified as Earthquake Prone Building (EPB), corresponding to a 'Grade D' building as defined by in the NZSEE building grading scheme. This is below the threshold for Earthquake Prone Buildings (34%NBS) as recommended by the NZSEE. Building No.21 has 10~25 times of risk relative to a new building, and considered to be "high risk" according to NZSEE grading system.

A Detailed Seismic Assessment is required in order to confirm the results from IEP, also to investigate deeply in the structural weaknesses.

Table 7- 10: Summary of IEP Assumptions and Justifications in Assessing Building No.21

IEP Item	Assumption	Justification
Date of Building Design	1965-1976	Before Capacity Design Theory introduced in design and construction
Soil Type	Type D	Soft soil
Building Importance Level	2	Public building
Ductility of Structure	2	Reinforced concrete frames and RC structural walls
Plan Irregularity (Factor A)	0.7	Severe (core wall systems with stairs and service loading on the east side)
Vertical Irregularity (Factor B)	0.7	Significant (rapid reinforcement details reduction)
Short Columns (Factor C)	0.7	Significant
Pounding (Factor D)	1	Insignificant
Site Characteristic (Factor E)	1	Insignificant
Factor F	0.8	Lower than 1 due to minimum transverse reinforcing and confinement details, and lack of capacity design

Table 7- 11: Summary of IEP Results of Building No.21

Building Name	**Securities House**				
Orthogonal Directions	**Potential Score**		**Potential Grade**	**Approximate Risk/New Building**	**Life-safety Risk Description**
Longitudinal	20%	EPB	D	10~25 times	high risk
Transverse	20%	EPB	D	10~25 times	high risk

7.3. Detailed Assessment of Building No.21 Frame 1

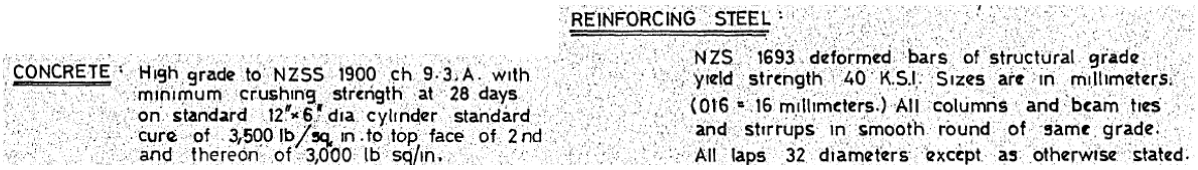
The current SLaMa procedure and the improved procedures (i.e. evaluation of strength hierarchy, determination of lower and upper bounds of lateral load capacity, and Portal Frame Method) were carried out for Building No.21 Frame 1. The assessment results computed from these procedures are shown in the following sections, and the differences in the results are discussed.

7.3.1. Determine Material Properties and Strengths

Table 7- 12: Summary of nominal material properties

Materials	Properties	Values
Concrete	Strength (fc') (MPa)	24.132 (≈24)
	Tensile Strength (fct) (MPa)	1.768
	Elastic Modulus (Ec) (MPa)	23209.160 (≈23209)
	Crushing Strain (εcu)	Assumed to be 0.003
Reinforcing Steel	Yield Strength (fsy) (MPa)	275.790 (≈276)
	Elastic Modulus (Es) (MPa)	200000
	Yield Strain (εsy)	0.001379
	Ultimate Strength (fsu) (MPa)	Assume 275.790 (≈276) (ignoring strain hardening effect, i.e. assuming bi-linear stress-strain relationship)
	Ultimate Strain (εsu)	0.15

Nominal material strengths of Building No.21 are clearly stated in the structural drawings, shown in Figure 7- 7, and summarised in Table 7- 12. Converting to S.I. units, the concrete strength (fc'), concrete tensile strength (assuming $f_{ct}=0.36(f_c')^{1/2}$), concrete elastic modulus (assuming $E_c=3320(f_c')^{1/2}+6900$), reinforcing steel yield strength, steel elastic modulus were calculated as 24.132MPa, 1.768MPa, 23209.16MPa, 275.790MPa, 200000MPa, respectively. The steel strain then was calculated out as 0.001379, and the crushing strain of concrete was assumed to be 0.003 due to inadequate confinement.


CONCRETE: High grade to NZSS 1900 ch 9.3.A. with minimum crushing strength at 28 days on standard 12"×6" dia cylinder standard cure of 3,500 lb/sq. in. to top face of 2nd and thereon of 3,000 lb sq/in.

REINFORCING STEEL: NZS 1693 deformed bars of structural grade yield strength 40 K.S.I. Sizes are in millimeters. (016 = 16 millimeters.) All columns and beam ties and stirrups in smooth round of same grade. All laps 32 diameters except as otherwise stated.

Figure 7- 7: Material Strengths Stated in Building No.21 Structural Drawings

The calculated probable material properties are shown in Table 7- 13, along with the approximation of potential varying ranges of the mean strengths. The use of nominal material properties listed in Figure 7- 7 provides a lower bound of component strength, while a use of probable properties of the materials gives an estimation of probable of component strength. For the cases where detailed material information is not available, probable strengths with variation of ±20% should be used in the assessment.

Table 7- 13: Summary of probable material properties and variation ranges

Materials	Properties	Values
Unconfined Concrete	Strength(fc') (MPa)	36.198($f'_{c,p} = 1.5 \times f'_c$) and [29, 43] (±20% of mean)
	Tensile Strength(fct) (MPa)	2.166
	Elastic Modulus(Ec) (MPa)	26874.705 (≈26875)
	Crushing Strain(εcu)	Assumed to be 0.003
Confined Concrete	Strength(fcc') (MPa)	54.297 ($f'_{cc} = 1.5 \times f'_{c,p}$) (≈54)
	Crushing Strain(εcu)	≈0.008~0.009 (for different sections) Mander Model $\rho_s = \frac{1.5A_v}{b_c s}$ and $\varepsilon_{cu} = 0.004 + \frac{1.4\rho_s f_{yh}\varepsilon_{su}}{f_{cc}}$, where: A_v = 157.08, 235.62, 314.16mm^2 (for different column sections) s = 229mm b_c = 457.2mm f_{yh} = 300MPa ε_{su} = 0.15

		f_{cc} = 54.297MPa
Reinforcing Steel	Yield Strength(fsy) (MPa)	297.853 (≈ 300) (1.08$fsy_{nominal}$) and [240, 360] (±20% of mean)
	Elastic Modulus(Es) (MPa)	200000
	Yield Strain(εsy)	0.0015
	Ultimate Strength(fsu)(MPa)	Assumed to be 300 (ignoring strain hardening effect)
	Ultimate Strain(εsu)	0.15

7.3.2. Determine Component Flexural Capacity

Based on the determined material properties and strengths, the flexural capacities of beams and columns were calculated following the procedure shown in Section 4.6.2.1. The calculated flexural strengths are summarised in Table 7- 14 and Table 7- 15.

It is worth noting that the presented strengths in the tables were determined based on the nominal material strength. These values define the lower bound of the component flexural capacity. The calculation based on the probable material strengths are shown in Section 7.4.1. For beam sections, the overstrength can be estimated as $M_{b,o} = 1.25 \times M_{b,n}$, giving the upper bound of the beam flexural capacity. The calculation of column overstrength requires more detailed consideration, such as the influence of axial load and confinement. For the columns confined by NZS3101:1995 specified amount of transverse reinforcement in potential plastic hinge regions, the overstrength can be calculated as $M_{col,o} = \left(1.25 + 2\left(\frac{N^*}{f_c' A_g} - 0.1\right)^2\right) M_{col,n} \geq 1.25 \times M_{col,n}$, while for the columns with less confining reinforcement, the overstrength can be calculated by applying the material overstrengths.

*Table 7- 14: Summary of beams and columns yielding and ultimate flexural moment capacity (based on nominal material strengths) (**without consideration of axial load on columns**) of Frame 1 in Building No.21*

Level	Beam				Column			
	Exterior		Interior		Exterior		Interior	
	ϕy	My	ϕy	My	ϕy	My	ϕy	My
	ϕu	Mu	ϕu	Mu	ϕu	Mu	ϕu	Mu
1	0.002809	323.94	0.002809	323.94	0.005104	300.16	0.005104	245.53
	0.054114	343.45	0.054114	343.45	0.035205	389.48	0.025514	348.91
2	0.002889	289.24	0.002679	214.24	0.005104	300.16	0.005104	245.53
	0.057150	301.48	0.067672	220.76	0.035205	389.48	0.025514	348.91
3	0.002889	289.24	0.002679	214.24	0.005104	300.16	0.005104	245.53
	0.057150	301.48	0.067672	220.76	0.035205	389.48	0.025514	348.91
4	0.002889	289.24	0.002679	214.24	0.005104	300.16	0.005104	245.53
	0.057150	301.48	0.067672	220.76	0.035205	389.48	0.025514	348.91
5	0.002590	160.85	0.002634	160.18	0.004411	130.09	0.004822	177.51
	0.054979	166.96	0.053515	166.98	0.076054	136.11	0.035205	260.08
6	0.002679	158.11	0.002737	156.79	0.004411	130.09	0.004411	130.09
	0.065930	164.64	0.058795	164.59	0.076054	136.11	0.076054	136.11
7	0.002679	158.11	0.002737	156.79	0.004411	130.09	0.004411	130.09
	0.065930	164.64	0.058795	164.59	0.076054	136.11	0.076054	136.11
R	0.002679	158.11	0.002737	156.79	0.004411	130.09	0.004411	130.09
	0.065930	164.64	0.058795	164.59	0.076054	136.11	0.076054	136.11

**unit: rad/m for curvature and kNm for moment*

*Table 7- 15: Summary of beams and columns yielding and ultimate flexural moment capacity (based on nominal material strengths) (**with consideration of axial load on columns**) of Frame 1 in Building No.21*

Level	Beam				Column			
	Exterior		Interior		Exterior		Interior	
	ϕy	My	ϕy	My	ϕy	My	ϕy	My
	ϕu	Mu	ϕu	Mu	ϕu	Mu	ϕu	Mu
1	0.002809	323.94	0.002809	323.94	0.006552	487.98	0.006296	386.78
	0.054114	343.45	0.054114	343.45	0.015441	513.58	0.016845	447.19
2	0.002889	289.24	0.002679	214.24	0.006336	459.33	0.006156	369.86
	0.057150	301.48	0.067672	220.76	0.017039	499.11	0.017745	436.30
3	0.002889	289.24	0.002679	214.24	0.006144	434.05	0.006015	352.73
	0.057150	301.48	0.067672	220.76	0.018617	484.89	0.018698	424.87
4	0.002889	289.24	0.002679	214.24	0.005948	408.39	0.005870	335.39
	0.057150	301.48	0.067672	220.76	0.020386	469.45	0.019706	412.96
5	0.002590	160.85	0.002634	160.18	0.005378	227.81	0.005545	254.88
	0.054979	166.96	0.053515	166.98	0.041196	244.14	0.022223	321.66
6	0.002679	158.11	0.002737	156.79	0.005143	202.85	0.005106	198.93
	0.065930	164.64	0.058795	164.59	0.049102	216.37	0.050360	211.96
7	0.002679	158.11	0.002737	156.79	0.004914	179.06	0.004886	176.17
	0.065930	164.64	0.058795	164.59	0.057126	189.69	0.058161	186.47
R	0.002679	158.11	0.002737	156.79	0.004664	154.08	0.004643	152.04
	0.065930	164.64	0.058795	164.59	0.066450	162.03	0.067246	159.80

In Table 7- 14, the column flexural strengths were calculated without taking axial loading into consideration, i.e. column strengths under "zero-axial load case". Also, it was found that G+ΨuQ-E≈0 for this case (especially for the upper levels), hence, it was assumed the flexural strengths calculated under zero axial loading are applicable under the G+ΨuQ-E (i.e. reverse earthquake loading combination) loading condition. In Table 7- 15, the column flexural strengths were calculated considering G+ΨuQ+E loading combination for exterior columns, and G+ΨuQ combination for interior columns. The details of axial loading are shown in Table 7- 16 and Table 7- 17.

Table 7- 16: Load combinations for exterior columns of Frame 1

Level	Cum area (m2)	G (kN)	Ψa	Q (kN)	ΨuQ (kN)	G +ΨuQ (kN)	1.2G +1.5Q (kN)	Beam Shears (kN)	E-induced axial load (kN)	G +ΨuQ+ E (kN)	G +ΨuQ- E (kN)
R	8.90	70.31	1.31	0.00	0.00	70.31	84.37	64.69	64.69	135.00	5.62
7	17.79	140.62	1.01	26.69	10.68	151.30	208.78	64.69	129.39	280.69	21.91
6	26.69	210.93	0.88	47.01	18.81	229.74	323.64	64.69	194.08	423.82	35.66
5	35.59	281.24	0.80	64.29	25.72	306.96	433.93	77.18	271.26	578.22	35.69
4	44.49	351.55	0.75	80.05	32.02	383.57	541.94	85.71	356.98	740.55	26.60
3	53.38	421.86	0.71	94.83	37.93	459.80	648.49	106.58	463.55	923.35	-3.76
2	62.28	492.18	0.68	108.92	43.57	535.74	753.99	106.58	570.13	1105.88	-34.39
1	71.18	562.49	0.66	122.49	49.00	611.48	858.72	133.99	704.12	1315.60	-92.64

Table 7- 17: Load combinations for interior columns of Frame 1

Level	Cum area (m2)	G (kN)	Ψa (0.5~1)	Q (kN)	ΨuQ (kN)	G+ΨuQ (kN)	1.2G+1.5Q (kN)
R	17.50	123.85	1.02	0.00	0.00	123.85	148.62
7	35.01	247.70	0.81	42.38	16.95	264.65	360.80
6	52.51	371.54	0.71	74.99	30.00	401.54	558.33
5	70.02	495.39	0.66	103.74	41.50	536.89	750.09
4	87.52	619.24	0.62	130.37	52.15	671.39	938.65
3	105.03	743.09	0.59	155.63	62.25	805.34	1125.15
2	122.53	866.93	0.57	179.92	71.97	938.90	1310.20
1	140.04	990.78	0.55	203.47	81.39	1072.17	1494.14

**q=3.0kPa and Ψu=0.4*

The seismic weight per floor was estimated to be 325 tones.

The bilinear moment-curvature curves for all the beams and columns of Frame 1 are shown in the following figures.

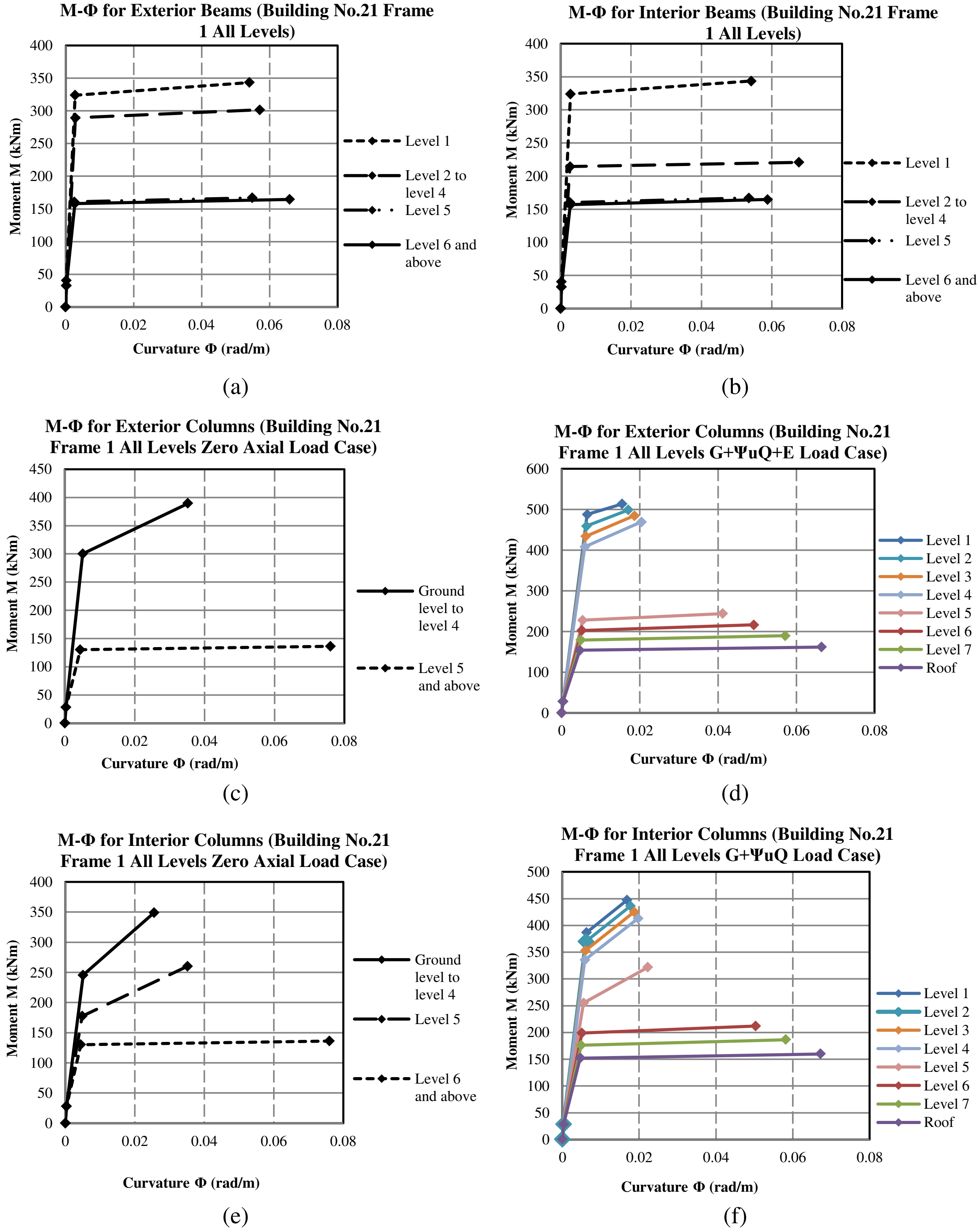


Figure 7- 8: Building No. 21 Frame 1 all levels: (a) Moment-curvature relationship for exterior beams; (b) Moment-curvature relationship for interior beams; (c) Moment-curvature relationship for exterior columns under zero axial load case (i.e. assumed to be the same for the G+ΨuQ-E load case); (d) Moment-curvature relationship for exterior columns under G+ΨuQ+E load case; (e) Moment-curvature relationship for interior columns under zero load case; (f) Moment-curvature relationship for interior columns under G+ΨuQ load case

7.3.3. Determine Component Shear Capacity and Demand (at Flexural Capacity)

Beam, column and joint shear capacities and demands were determined following the procedure shown from Section 4.6.2.2 to Section 4.6.2.4.

In Table 7- 18 and Table 7- 19, the calculated beam shear capacities and demands are shown. It is worth noting that the least and the largest value of k, i.e. 0.05 and 0.2 respectively, were used to calculate the lower and upper bounds of the beam shear capacities. It was found that V_{BPI} is greater than V_{BD}, indicating that beam shear capacities meet the required demands, and beam shear failure was not be expected.

Table 7- 18: Exterior beam shear capacity calculation for Frame 1 of Building No.21

Level	fc' (Mpa)	Bw (mm)	d_{beam} (mm)	fyt (Mpa)	s (mm)	d_s (mm)	Asp (mm^2)	Av (mm^2)	Lbc (m)	wb (kN/m)	VBG (kN)	VBPI k=0.2 (kN)	VBPI k=0.05 (kN)	VBD (kN)
R (8)	24.1	203.2	685.8	275.8	177.8	10	78.5	157.1	6.6	4.4	14.4	258.4	171.1	64.7
7	24.1	203.2	685.8	275.8	177.8	10	78.5	157.1	6.6	4.4	14.4	258.4	171.1	64.7
6	24.1	203.2	685.8	275.8	177.8	10	78.5	157.1	6.6	4.4	14.4	258.4	171.1	64.7
5	24.1	203.2	685.8	275.8	177.8	10	78.5	157.1	6.6	4.4	14.4	258.4	171.1	65.3
4	24.1	203.2	685.8	275.8	177.8	10	78.5	157.1	6.6	4.4	14.4	258.4	171.1	106.6
3	24.1	203.2	685.8	275.8	177.8	10	78.5	157.1	6.6	4.4	14.4	258.4	171.1	106.6
2	24.1	203.2	685.8	275.8	177.8	10	78.5	157.1	6.6	4.4	14.4	258.4	171.1	106.6
1	24.1	254.0	685.8	275.8	177.8	10	78.5	157.1	6.6	4.4	14.4	287.5	178.4	118.9

Table 7- 19: Interior beams shear capacity calculation for Frame of Building No.21

Level	fc' (Mpa)	Bw (mm)	d_{beam} (mm)	fyt (Mpa)	s (mm)	d_s (mm)	Asp (mm^2)	Av (mm^2)	Lbc (m)	wb (kN/m)	VBG (kN)	VBPI k=0.2 (kN)	VBPI k=0.05 (kN)	VBD (kN)
R (8)	24.1	203.2	685.8	275.8	177.8	10	78.5	157.1	6.3	4.4	13.9	258.4	171.1	54.0
7	24.1	203.2	685.8	275.8	177.8	10	78.5	157.1	6.3	4.4	13.9	258.4	171.1	54.0
6	24.1	203.2	685.8	275.8	177.8	10	78.5	157.1	6.3	4.4	13.9	258.4	171.1	54.0
5	24.1	203.2	685.8	275.8	177.8	10	78.5	157.1	6.3	4.4	13.9	258.4	171.1	66.3
4	24.1	203.2	685.8	275.8	177.8	10	78.5	157.1	6.3	4.4	13.9	258.4	171.1	83.7
3	24.1	203.2	685.8	275.8	177.8	10	78.5	157.1	6.3	4.4	13.9	258.4	171.1	83.7
2	24.1	203.2	685.8	275.8	177.8	10	78.5	157.1	6.3	4.4	13.9	258.4	171.1	83.7
1	24.1	254.0	685.8	275.8	177.8	10	78.5	157.1	6.3	4.4	13.9	287.5	178.4	122.2

In Table 7- 20 and Table 7- 21, the calculated column shear capacities and demands are shown. As specified in Section 5.3.2, several assumptions were made in order to simplify the calculation process, for instance, ignoring the shear resistance contributed by the axial compression load N*. Like in computing the beam shear capacity, the least and the largest value of k, i.e. 0.1 and 0.29, were applied to calculate the lower and upper bounds of the shear resisted by concrete mechanism. It was found that the level 1 exterior column shear capacities do not meet the required demands; thus, it was predicted that exterior column shear failure might occur at the bottom level in a seismic event. For the interior columns, when k=0.1 was applied, it was found that the level 1 interior column shear capacity do not meet the demand, hence, column shear failure was also expected to occur at the bottom level in a seismic event. However, it was calculated that the curvature ductility factors are within the range [2.6, 3.4] for the lower level interior columns. By referring to Figure 4- 5 in Section 4.6.2.4, the value of k was estimated to be within the range of [0.176, 0.214] considering biaxial loading and [0.271, 0.29] considering only uniaxial loading. Hence, when only uniaxial loading was considered, the capacity was calculated to be greater than the demand, and no column shear failure was expected. When biaxial loading was considered, the capacity was found to be insufficient, and column shear failure was expected.

Table 7- 20: Exterior column shear capacity calculation for Frame 1 of Building No.21

Level	bc mm	hc mm	ρl	α	β	k	Vc kN	s mm	d_s mm	Asp mm^2	Av mm^2	d"$_c$ mm	Vs kN	Vn kN	VCPI kN	Lc m	VCD kN	VCD no ωvk N
7-R (8)	457	457	0.012	1	0.74	0.21	175	229	10	78.5	157.1	419	138	0	225	3.048	63	54
6-7	457	457	0.012	1	0.74	0.21	175	229	10	78.5	157.1	419	138	0	225	3.048	63	54
5-6	457	457	0.012	1	0.74	0.21	175	229	10	78.5	157.1	419	138	0	225	3.048	64	55
4-5	457	457	0.012	1	0.74	0.21	175	229	10	78.5	157.1	419	138	0	225	3.048	90	77
3-4	457	457	0.035	1	1	0.29	238	229	10	78.5	235.6	419	206	0	320	3.048	115	99
2-3	457	457	0.035	1	1	0.29	238	229	10	78.5	235.6	419	206	0	320	3.048	115	99
1-2	457	457	0.035	1	1	0.29	238	229	10	78.5	235.6	419	206	0	320	3.048	123	106
G-1	457	457	0.035	1	1	0.29	238	229	10	78.5	235.6	419	206	0	320	3.048	393	337

k	VCPI kN	VCD kN	VCD noωv kN
0.1	158	63	54
0.1	158	63	54
0.1	158	64	54
0.1	158	90	77
0.1	208	115	99
0.1	208	115	99
0.1	208	123	106
0.1	208	393	337

Table 7- 21: Interior column shear capacity calculation for Frame 1 of Building No.21

Level	bc mm	hc mm	ρl	α	β	k	Vc kN	S mm	d_s mm	Asp mm^2	Av mm^2	d"$_c$ mm	Vs kN	Vn kN	VCPI kN	Lc m	VCD kN	VCD no ωvkN
7-R (8)	457	457	0.0118	1	0.74	0.21	175	228.6	10	78.5	157.1	419	138	0	225	3.048	126	108
6-7	457	457	0.0118	1	0.74	0.21	175	228.6	10	78.5	157.1	419	138	0	225	3.048	126	108
5-6	457	457	0.0118	1	0.74	0.21	175	228.6	10	78.5	157.1	419	138	0	225	3.048	127	96
4-5	457	457	0.0236	1	0.97	0.28	231	228.6	10	78.5	157.1	419	138	0	266	3.048	164	167
3-4	457	457	0.0353	1	1.00	0.29	238	228.6	10	78.5	314.2	419	275	0	370	3.048	201	225
2-3	457	457	0.0353	1	1.00	0.29	238	228.6	10	78.5	314.2	419	275	0	370	3.048	201	225
1-2	457	457	0.0353	1	1.00	0.29	238	228.6	10	78.5	314.2	419	275	0	370	3.048	231	241
G-1	457	457	0.0353	1	1.00	0.29	238	228.6	10	78.5	314.2	419	275	0	370	3.048	342	293

k	VCPI kN	VCD kN	VCD noωv kN
0.1	158	126	108
0.1	158	126	108
0.1	158	127	96
0.1	158	164	167
0.1	257	201	225
0.1	257	201	225
0.1	257	251	241
0.1	257	342	293

In Table 7- 22 and Table 7- 23, the calculated joint shear capacities and demands are shown. The least and the largest value of k, i.e. 0.3 and 0.4 for exterior joints, 0.3 and 1.0 for interior joints, were applied to compute the lower and upper bounds of the shear capacities. For the exterior joints, it was found that under N(G+ΨuQ-E) and N(=0) axial load conditions, the lower level (i.e. generally level 1 to 3) joint shear capacities do not meet the required demands; hence, it was expected that joint shear failure might occur at lower levels under these loading conditions. For the interior joints, at the lower levels, the curvature ductility factors were found to be within the range [2.6, 3.4], and the k value was calculated to be within the range of [0.86, 0.94] referring to Figure 4- 4 in Section 4.6.2.3. It was found that the joint shear capacity meets the demand at this condition. Hence, no joint shear failure was predicted for the interior joints. It is worth noting that this conclusion is not consistent with the results from the evaluation of strength hierarchy shown in the following section, which requires further investigation and modification of the current SLaMa procedure associated with the determination of joint shear capacity.

Table 7- 22: Exterior joint shear capacity calculation for Frame 1 of Building No.21

Level	fc' Mpa	bj mm	hc mm	Ag mm2	Lb mm	Lbc mm	hb mm	Lc mm	N*(G +ΨQ) kN	Vpji k=0.4 kN	Vpji k=0.3 kN	ΣMb kNm	Vijk N
R (8)	24.1	457.2	457.2	209032	7.0	6.6	736.6	3048	70	378	290	165	191
7	24.1	457.2	457.2	209032	7.0	6.6	736.6	3048	151	408	320	165	191
6	24.1	457.2	457.2	209032	7.0	6.6	736.6	3048	230	436	346	165	191
5	24.1	457.2	457.2	209032	7.0	6.6	736.6	3048	307	462	370	167	193
4	24.1	457.2	457.2	209032	7.0	6.6	736.6	3048	384	486	392	301	348
3	24.1	457.2	457.2	209032	7.0	6.6	736.6	3048	460	508	413	301	348
2	24.1	457.2	457.2	209032	7.0	6.6	736.6	3048	536	530	433	301	348
1	24.1	457.2	457.2	209032	7.0	6.6	736.6	3048	611	551	452	343	397

N*(G+ ΨQ+E) kN	Vpji k=0.4 kN	Vpji k=0.3 kN	Vijk N
136	403	314	191
282	453	362	191
426	498	404	191
580	542	445	193
743	585	484	348
927	630	524	348
1110	672	562	348
1320	717	602	397

N*(G+ ΨQ-E) kN	Vpji k=0.4 kN	Vpji k=0.3 kN	Vij kN
5	351	264	191
21	358	271	191
34	363	276	191
33	363	276	193
24	359	272	348
-7	346	259	348
-38	332	245	348
-97	305	217	397

N	Vpji k=0.4 kN	Vpji k=0.3 kN	Vij kN
0	349	262	191
0	349	262	191
0	349	262	191
0	349	262	193
0	349	262	348
0	349	262	348
0	349	262	348
0	349	262	397

Table 7- 23: Interior joint shear capacity calculation for Frame 1 of Building No.21

Level	fc' Mpa	bj mm	hc mm	Ag mm2	Lb mm	Lbc mm	hb mm	Lc mm	N*(G +ΨQ) kN	Vpji k=1.0 kN	Vpji k=0.3 kN	ΣMb kNm	Vijk N
R (8)	24.1	457.2	457.2	209032	6.8	6.3	736.6	3048	124	924	310	330	382
7	24.1	457.2	457.2	209032	6.8	6.3	736.6	3048	265	979	357	330	382
6	24.1	457.2	457.2	209032	6.8	6.3	736.6	3048	402	1029	397	330	382
5	24.1	457.2	457.2	209032	6.8	6.3	736.6	3048	537	1077	434	334	386
4	24.1	457.2	457.2	209032	6.8	6.3	736.6	3048	671	1122	467	522	604
3	24.1	457.2	457.2	209032	6.8	6.3	736.6	3048	805	1166	498	522	604
2	24.1	457.2	457.2	209032	6.8	6.3	736.6	3048	939	1208	527	522	604
1	24.1	457.2	457.2	209032	6.8	6.3	736.6	3048	1072	1248	554	686	793

7.3.4. Evaluate Strength Hierarchy at Local Level (i.e. subassembly level)

In Figure 7- 9, the results of strength hierarchy evaluation are shown for all the joints in Frame 1. It was predicted that the exterior joints and the interior joints at higher levels follow the sequence "beam flexural hinging – joint shear failure – column flexural hinging" in general, while for the interior joints at level 1, level 3 and level 4, the joints tend to have joint shear failure before the occurrence of beam and column flexural hinging, in other words, these joints tend to have the sequence "joint shear failure – beam flexural hinging – column flexural hinging".

In Frame 1 of Building No.21, joints A1 and D1 are corner joints, and, joints B1 and C1 are one-way interior joint. It can be expected that "corner joints might represent the critical conditions in building frames because of the biaxial input, typically difficult reinforcement detailing problems involved in anchoring two orthogonal sets of beam bars in the joint, and the influence of variable axial load. Despite this concern, there are almost no test data available for this type of joint" (*Priestley, 1997*). It was indicated in the structural drawings that the deformed transverse reinforcing bars of beams and columns were bent into joint core. Hence, it was assumed that a reliable compression strut should develop in the joint core, and the principle tensile stress was assumed to be $0.42\sqrt{fc'}$ for the exterior corner joints (e.g. A1, D1) and one-way joint (e.g. B1, C1). It is worth noting that in Frame 1, there is no "true" interior joint, as B1 and C1 are exterior joints along the orthogonal direction; thus the k value of 0.42 was applied also to B1 and C1 for conservative purpose.

The calculation was carried out following the specification in Section 5.3.3 and Section 5.4.

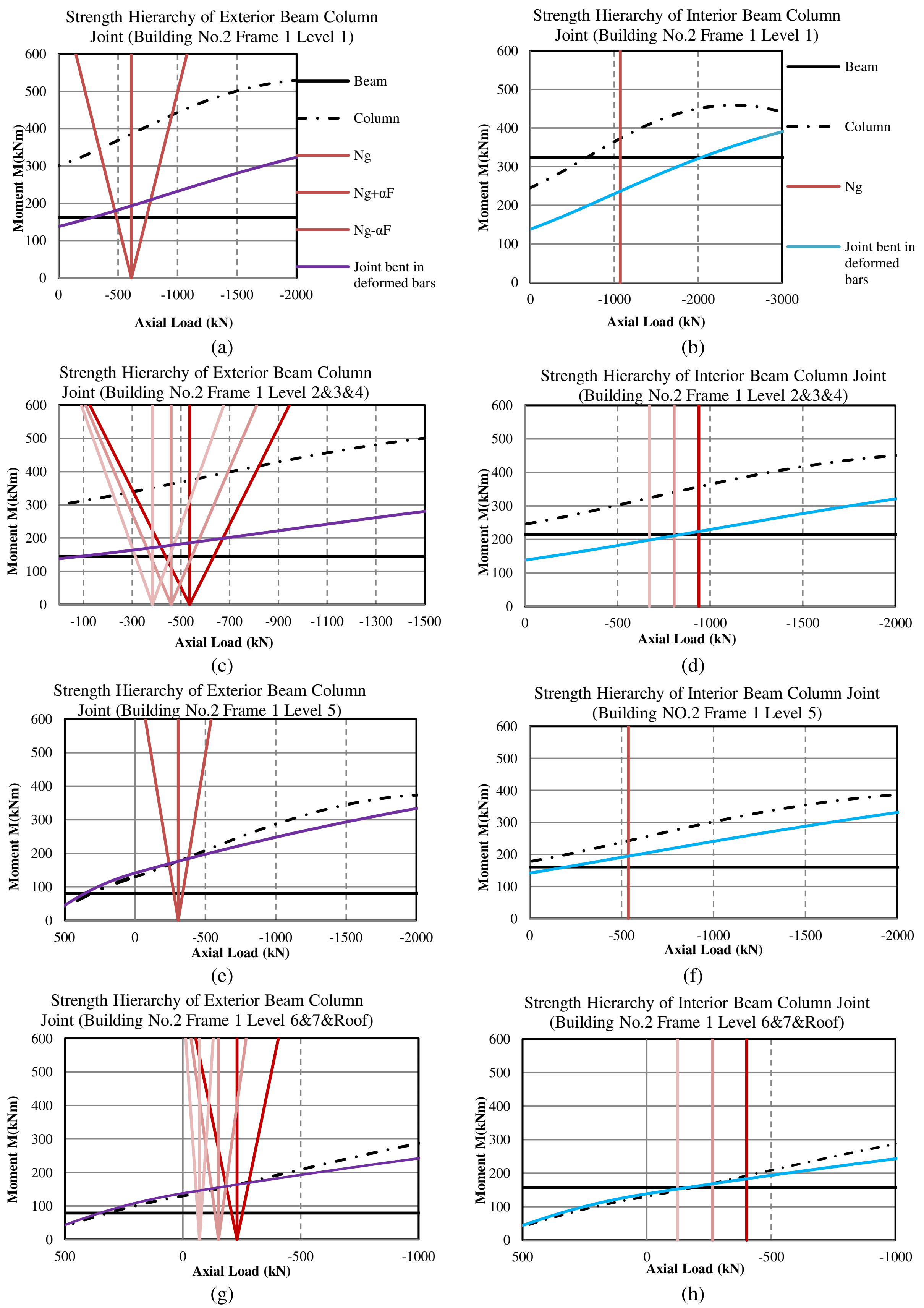


Figure 7- 9: Strength hierarchy evaluation for joints in Frame 1 of Building No.21: (a) Exterior joints at level 1; (b) Interior joints at level 1; (c) Exterior joints at level 2&3&4; (d) Interior joints at level 2&3&4; (e) Exterior joints at level 5; (f) Interior joints at level 5; (g) Exterior joints at level 6&7&Roof; (h) Interior joints at level 6&7&Roof

7.3.5. Determine Global Mechanism

To determine global mechanism, as specified from Section 5.5 to Section 5.6, (1) the current SLaMa; (2) the improved SLaMa with evaluation of strength hierarchy and determination of lower and upper bounds of lateral load capacity; and (3) the improved SLaMa with evaluation of strength hierarchy and Portal Frame Method; were applied to assess Frame 1 of Building No.21. As for the adoption of component analysis models and global structure models, due to timeframe constraint and scope of thesis, only discussion regarding the application of such models are provided in this section.

Table 7- 24 and Table 7- 25 provide summaries of differences in the calculation processes of the four methods, with descriptions and the computed results shown. More detailed explanations regarding each of the methods are shown from Section 7.3.5.1 to Section 7.3.5.4, and in Section 7.3.5.5, a comparison among the pushover curves computed from the four methods is shown. In Section 7.3.5.6, a comparison between the assessment results and the observed structural damages is presented.

Table 7- 24: Comparison of the procedures (in description and without numbers)

Calculation Procedure	**Current SLaMa in NZSEE 2006**		**Improved SLaMa with Evaluation of Strength Hierarchy and Determination of Lower and Upper Bounds of Lateral Load Capacity**	**Improved SLaMa with Evaluation of Strength Hierarchy and Portal Frame Method**	**Adoption of Component Analysis Models and Global Structure Models**
FB or DB	FB	DB	DB	DB	Not applicable
Predicted Mechanism	Beam sidesway	Beam sidesway	Beam sidesway: upper bound Column sidesway: lower bound	Mixed sidesway	Mixed sidesway
Displaced shape required	No	From DDBD	Procedures from: DDBD Priestley Seismic Design Book	From Portal Frame Method calculation	Not applicable
Effective Height H_{eff}	Current SLaMa instructions (Section 4.6.2.6) and instructions from NZSEE 2006 Section 7.2.4.		Based on the displaced shape calculated following the procedure in Priestley Seismic Design Book and DDBD, $h_{eff} = \frac{\sum m_i \Delta_i h_i}{\sum m_i \Delta_i}$	Based on the displaced shape calculated in Portal Frame Method	Not applicable
Contribution of $\Sigma(N_{Ei}L_i)$ to the total overturning moment	Sum of all end beam shears (calculated based on beam flexural capacities)		For the beam sidesway: Sum of all end beam shears (calculated based on beam flexural capacities) For the column sidesway: Sum of the end beam shears of the levels below the level where column sway is expected. For Frame 1 in Building No.21, it should be the sum of the end beam shears from level 1 to 4, as the column sway mechanism is expected to occur at level 5	Sum of the beam shears depending on the mechanism. For Frame 1 in Building No.21, at level 1, 3, and 4, the interior joint shear failure was predicted to occur before occurrence of exterior beam hinging. Therefore, the beam shears at level 1, 3, and 4 were calculated based on the joint equivalent moment. Also, since it was predicted that column sway mechanism occurs at level 5, the contribution from the beam shears above level 5 was ignored.	Should be determined depending on the mechanism, and referring to the selected structural representatives
Contribution of ΣM_{coi} to the total overturning moment	Sum of moments for column at ground levels and the moments should be determined based on beam flexural capacities		SAME as left	SAME as left	SAME as left, and should refer to the selected component representatives
Yielding displacement Δ_y	Current SLaMa instructions (Section 4.6.2.7)		Procedure in Priestley Seismic Design Book	SAME as left	Approximated from the computed pushover curve directly
Ultimate displacement Δ_u	Current SLaMa instructions (Section 4.6.2.9)		Procedure in DB, $\Delta_u = \frac{\sum m_i \Delta_i^2}{\sum m_i \Delta_i}$	SAME as left	Approximated from the computed pushover curve directly
Ductility capacity μ_{sc}	From the guidelines shown in NZSEE 2006 Figure 4E.8 (shown in Section 4.6.2.8 Figure 4-6) Or should be determined by the formula provided in NZSEE 2006 7.2.4 (summarised in Section 4.6.2.8 Table 4- 21)		Determined from $\mu_{sc} = \frac{\Delta_u}{\Delta_y}$	Determined from $\mu_{sc} = \frac{\Delta_u}{\Delta_y}$	SAME as left

Table 7- 25: Comparison of the procedures (with numbers and showing the percentage of the differences)

Calculation Procedure	**Current SLaMa in NZSEE 2006**	**Improved SLaMa with Evaluation of Strength Hierarchy and Determination of Lower and Upper Bounds of Lateral Load Capacity**	**Improved SLaMa with Evaluation of Strength Hierarchy and Portal Frame Method**	**Adoption of Component Analysis Model and Global Structure Model**
FB or DB	FB | DB	DB	DB	Not applicable
Predicted Mechanism	Beam sidesway | Beam sidesway	Beam sidesway: upper bound Column sidesway: lower bound	Mixed sidesway	Mixed sidesway
Displaced shape required: ratio of displacement at each level to the top displacement	(FB: —) DB: DBD (BS) 1 0.91 0.81 0.70 0.58 0.45 0.31 0.16	DBD (BS) | Design (BS) | Design (5CS) 1 | 1 | 1 0.91 | 0.94 | 0.93 0.81 | 0.88 | 0.85 0.70 | 0.74 | 0.76 0.58 | 0.69 | 0.40 0.45 | 0.61 | 0.30 0.31 | 0.53 | 0.21 0.16 | 0.43 | 0.10	PFM (5InCol) | PFM (5InCol) 1 | 1 0.96 | 0.91 0.88 | 0.82 0.62 | 0.61 0.32 | 0.36 0.25 | 0.27 0.18 | 0.18 0.11 | 0.09	Not applicable
Effective Height H_{eff}	Current SLaMa (BS) m | Current NZSEE (BS) m 17.27 | 14.39	DBD (BS) m | Design (BS) m | Design (5CS) m 16.82 | 15.52 | 17.69	PFM (5InCol) m | PFM (5InCol) m 18.10 | 18.00	Not applicable
Contribution of $\Sigma(N_{Ei}L_i)$ to the total overturning moment	BS (kNm) 14862	BS (kNm) | 5CS (kNm) 14862 | 9456	MS (kNm) 10038	Not applicable
Contribution of ΣM_{coi} to the total overturning moment	BS (kNm) 686	BS (kNm) | 5CS (kNm) 686 | 686	MS (kNm) 686	Not applicable
Base shear capacity	Current SLaMa (BS) (kN) | Current NZSEE (BS) (kN) 900 | 1081	DBD (BS) (kN) | Design (BS) (kN) | Design (5CS) (kN) 924 | 1002 | 573	PFM (5InCol) (kN) | PFM (yield assumption) (5InCol) (kN) 593 | 596	Not applicable
Yielding displacement Δ_y (at H_{eff} and at top)	Current SLaMa (BS) (mm) | Current NZSEE (BS) (mm) 109.85 | 91.50 155.08 | 155.08	DBD (BS) (mm) | Design (BS) (mm) | Design (5CS) (mm) 51.14 | 47.73 | 53.40 69.80 | 69.80 | 69.80	PFM (5InCol) (mm) | PFM (yield assumption) (5InCol) (mm) 54.47 | 54.21 69.80 | 69.80	Not applicable
Ultimate displacement Δ_u	Current SLaMa (BS) (mm) | Current NZSEE (BS) (mm) 402.00 | 383.65 567.52 | 650.25 Column shear failure (mm) | Joint shear failure (mm) 109.85 | 219.69 155.08 | 310.16	DBD (BS) (mm) | Design (BS) (mm) | Design (5CS) (mm) 216.64 | 94.96 | 73.82 292.68 | 122.27 | 96.98	PFM (5InCol) (mm) | PFM (yield assumption) (5InCol) (mm) 73.06 | 82.64 95.71 | 112.52	Not applicable
Ductility capacity μ_{sc}	Current SLaMa (BS) | Current NZSEE (BS) 1, 2, 3.67 | 4.19	DBD (BS) | Design (BS) | Design (5CS) 4.24 | 1.99 | 1.38	PFM (5InCol) | PFM (yield assumption) (5InCol) 1.34 | 1.52	Not applicable

*(*NOTE: μ_{sc} =1, 2, 3.67 corresponding to different ultimate displacements)*

7.3.5.1. Current SLaMa

As explained in Table 7- 24, the detailed calculation procedure is shown in Section 4.6, and the calculation results are recorded in Table 7- 25.

Based on the calculated joint sway potentials that are summarised in Table 7- 26, it was expected that hinges might form at the top or bottom of the interior joint regions at level 1, 4, and 7 (i.e. S_{pij}>0.85). However, the current SLaMa does not include clarified instructions to take the joint sway potential (i.e. "weaker column than beam") into consideration. Since the storey sway potentials for all levels were found to be less than 0.85, beam sidesway mechanism was predicted.

Table 7- 26: Summary of joint sway potential and storey sway potential

Level	**Spij (sway potential at joint on column i at level j)**				**Spjk (Storey Sway Potential)**
	Exterior Joint A1	**Interior Joint B1**	**Interior Joint C1**	**Exterior Joint D1**	
7	0.47	0.95	0.73	0.47	**0.65**
6	0.41	0.83	0.64	0.41	**0.57**
5	0.36	0.62	0.63	0.36	**0.50**
4	0.42	0.71	0.93	0.43	**0.63**
3	0.32	0.63	0.82	0.32	**0.51**
2	0.31	0.61	0.80	0.31	**0.49**
1	0.34	0.89	0.89	0.44	**0.62**

It was predicted that for the ground level columns and the lower level exterior joints, the shear capacities are insufficient to meet the required demands; hence, column shear failure or joint shear failure were expected to occur at the bottom or other lower level. According to Figure 4- 6 in Section 4.6.2.8 (i.e. NZSEE 2006 Figure 4E.8), the ductility capacity was estimated to be 1 for column shear failure situation, 2 for joint shear failure situation and 3.67 calculated for beam hinging mechanisms. Therefore, the ultimate displacements estimated were different depending on the ductility determined, as shown in Figure 7- 10.

The pushover curves computed by applying the procedure stated in the current SLaMa and NZSEE 2006 are shown in Figure 7- 10. The findings listed below confirm the limitations or shorcomings of the current SLaMa mentioned in previous chapters, hence, confirm the need of modification and improvement.

- The determination of effective height is not clarified: NZSEE 2006 Section 7.2.4 and SLaMa procedure from NZSEE 2006 Appendix 4E provide different approaches to determine effective height. The formula to determine effective height of a frame structure from NZSEE 2006 Section 7.2.4 (see Section 4.6.2.6) gave a lower estimation, hence, a higher lateral load capacity was computed.
- No difference in the computed results was found by applying force-based approach and displacement-based approach. This is due to the reason that the current SLaMa lacks clarified instructions of determining displaced shape in displacement-base approach; hence, the

effective height was not properly estimated. The same effective height computed in force-based approach was applied when using displacement-based approach.

- The base shear capacities corresponding to different mechanisms were found to be the same. This is due to the reason that the current SLaMa lacks correct estimation of the axial load induced overturning moment contribution to the overall overturning moment.
- The displacement capacities were found to be overestimated.

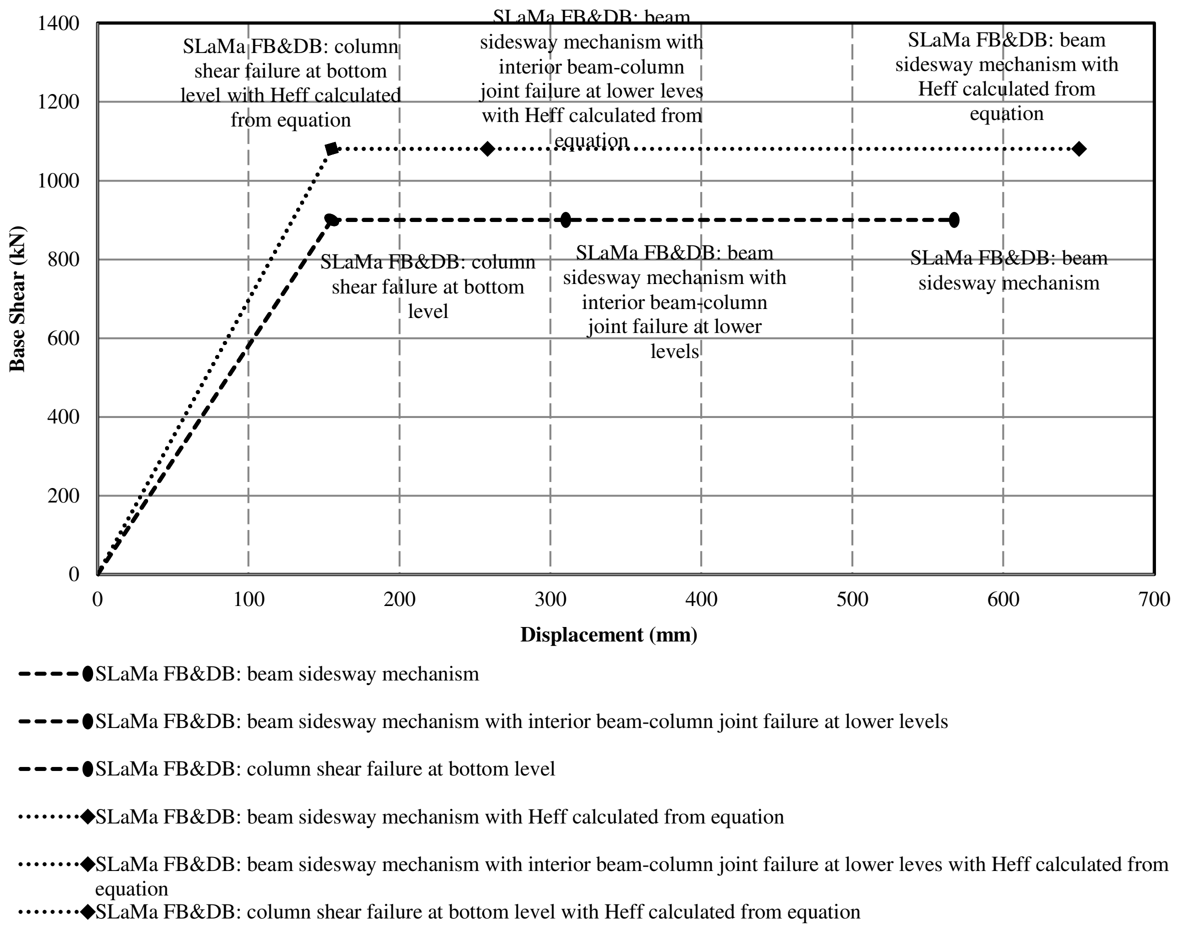


Figure 7- 10: Building No.21 Frame 1 pushover curve by the current SLaMa method

7.3.5.2. Determine Lower and Upper Bounds of Lateral Load Capacity

The detailed calculation procedure to determine lower and upper bounds of lateral load capacity is shown in Section 5.5.1, and the calculation results are recorded in Table 7- 25.

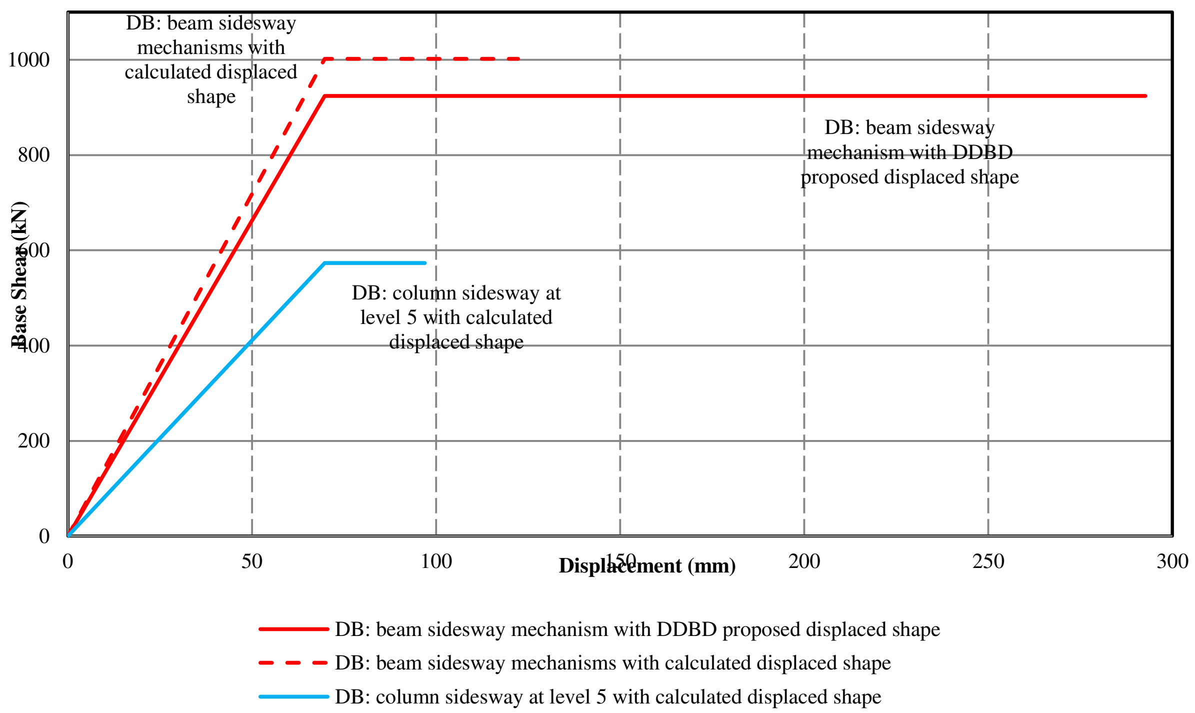


Figure 7- 11: Building No.21 Frame 1 pushover curve upper and lower bounds by the improved SLaMa

The computed lower (i.e. blue) and upper (red) bounds are shown in Figure 7- 11, and the findings are listed as following:

- The adoption of displaced-shape of a frame with beam sidesway mechanism from DDBD gave a lower lateral load capacity and higher ultimate displacement (i.e. the red straight curve), compared to the results generated from the procedure in *Seismic Design of Reinforced Concrete and Masonry Building (Paulay, T. and Priestley, M.J.N.)* (i.e. the red dot line). The displacements for all levels, corresponding to the two mechanisms – beam sidesway mechanism and column sidesway mechanism at level 5, were calculated from the *Seismic Design of Reinforced Concrete and Masonry Building* procedure. The calculated displacements are recorded in the following table (Table 7- 27).

Table 7- 27: Calculated displacements of all levels for beam sidesway and column sidesway mechanism

Level	Displacement from the *Design* procedure (BS-beam sidesway mechanism) (m)	Displacement from the *Design* procedure (5CS-column sidesway mechanism at level 5) (m)
8	0.122	0.097
7	0.115	0.090
6	0.107	0.082
5	0.090	0.074
4	0.085	0.039
3	0.074	0.029
2	0.065	0.020
1	0.053	0.010

- The yielding displacement of the frame was calculated by $\Delta_y = \frac{l_c^2}{6}\sum_1^n \varphi_{yci}$=69.8mm at the top level, independent of mechanisms.
- Based on the understanding of the structure, it was expected that the frame was most likely to subject to column sway mechanism at level 5 with joints failure at lower levels. Therefore, it was predicted that the pushover over curve should be within the range of upper and lower bounds, and should be more close to the lower bound.

7.3.5.3. Determine Sequence of Mechanisms by Applying Portal Frame Method

Following the procedure stated in Section 5.5.2, the sequence of mechanisms determined for Frame 1 of Building No.21 is recorded in Table 7- 28.

Table 7- 28: Sequence of mechanisms determined by Portal Frame method

Level	Mechanism	Storey Shear (kN)	Base Shear (kN)	Local displacement (mm)	Global displacement (mm)	Global displacement with yield state assumption (mm)
Level 5	Beam yield (structure "First Yield" state)	313	434	2.94	23.52	
Level 2	Interior beam flexural hinging	437	449	5.69	26.27	
Level 5	Exterior beam flexural hinging	325	450	3.05		
Level 5	Interior beam flexural hinging	331	458	5.63	28.95	
Level 3	Interior beam flexural hinging	437	477	5.83	31.84	
Level 4	Interior beam flexural hinging	437	524	5.98	34.89	
Level 6	Beam yield	308	528	3.10		
Level 6	Exterior beam flexural hinging	321	550	3.23		
Level 6	Interior beam flexural hinging	326	558	6.54	38.49	
Level 2	Beam yield	563	579	3.09		
Level 2	Exterior beam flexural hinging	587	604	3.22		
Level 2	Joint shear failure	594	611			
Level 3	Beam yield	563	614	3.17		
Level 3	Joint shear failure	567	619			
Level 1	Joint shear failure	620	620		41.38	
Level 1	Beam yield	631	631	3.37		
Level 3	Exterior beam flexural hinging	587	640	3.30		
Level 4	Joint shear failure	539	646		42.63	
Level 1	Exterior beam flexural hinging	669	669	3.57		
Level 4	Beam yield	563	676	3.26		
Level 1	Interior beam flexural hinging	680	680	8.66	44.20	
Level 4	Exterior beam flexural hinging	587	704	3.40		
Level 6	Interior column flexural hinging	420	719	20.91	58.57	
Level 5	Joint shear failure	520	721		58.66	
Level 7	Beam yield	308	739	3.36		
Level 7	Exterior beam flexural hinging	321	769	3.50		
Level 7	Interior beam flexural hinging	326	782	7.07	62.70	71.52
Level 6	Joint shear failure	476	816		69.18	
Level 5	**Interior column flexural hinging**	**637**	**882**	**24.60**	**81.67**	**99.14**
Level 1	Interior column flexural hinging	885	885	17.15	90.16	107.35
Level 7	Interior column flexural hinging	369	886	21.26	104.36	119.68
Level 2	Interior column flexural hinging	864	888	18.45	117.12	129.19
Level 3	Interior column flexural hinging	841	917	19.91	124.62	140.16
Level 4	Interior column flexural hinging	817	981	21.57	140.21	152.80
Level 7	Joint shear	441	1057			
Level 5	Exterior column flexural hinging	951	1316	19.62		
Level 5	Column Shear failure	982	1359			
Level 1	Column Shear failure	1379	1379			
Level R	Beam yield	308	1385	3.71		
Level 2	Column Shear failure	1379	1419			
Level R	Interior column flexural hinging	316	1423	21.32		
Level R	Exterior beam flexural hinging	321	1443	3.86		
Level 6	Exterior column flexural hinging	843	1444	20.81		

Level R	Interior beam flexural hinging	326	1466	0.00		
Level 3	Column Shear failure	1379	1505			
Level 6	Column Shear failure	901	1544			
Level 4	Column Shear failure	1379	1655			
Level 7	Exterior column flexural hinging	739	1773	21.24		
Level R	Joint shear joint	401	1804			
Level 2	Exterior column flexural hinging	1944	1999	14.37		
Level 1	Exterior column flexural hinging	2000	2000	12.65		
Level 3	Exterior column flexural hinging	1888	2060	16.21		
Level 7	Column Shear failure	901	2162			
Level 4	Exterior column flexural hinging	1828	2194	18.49		
Level R	Exterior column flexural hinging	631	2840	21.32		
Level R	Column Shear failure	901	4054			

(*NOTE*: The deformation due to shear type of mechanisms was not appropriately estimated. In order to plot the lateral loads corresponding to the shear mechanisms, linear interpolation was applied to approximate the displacements.)*

Appendix A11 provides summaries of calculation of global displacement and displaced ratio following the sequence of mechanisms determined from Portal Frame Method. Figure 7- 12 and Figure 7- 13 illustrate the development of displaced shapes following the sequence of mechanisms, and the computed pushover curves with the sequence of mechanisms are shown in Figure 7- 14 (i.e. light green and dark green polygonal lines).

From the computed results, the sequence of mechanisms for Building No.21 Frame 1 can be described as: beam flexural hinging – joint shear damage – interior column flexural hinging – exterior column flexural hinging – column shear damage. It is worth noting that not all these mechanisms can develop, and the joint shear damage or/and interior column flexural hinging may trigger partial or total failure of the structure before forming of the later mechanisms. The determination of the potential failure mechanism is vital; otherwise, the lateral load capacity may be overestimated. For this frame, it was assumed that the mechanism "Level 5 Interior Column Hinging" (the largest interstorey drift calculated corresponding to this mechanism, but was found to be less than ULS of 2.5%) might lead to a partial failure of the structural. The displaced shape calculated corresponding to this mechanism was applied in the displacement-based assessment approach to compute a bilinear pushover curve, shown in Figure 7- 14 (i.e. light green and dark green bi-linear curves). However, it has been discovered that the application of the displaced shapes provided lower base shear capacities but higher displacement capacities. It can be inferred that the bi-linear curves should be more close to the actual response of the structures, for the reasons listed below:

- In calculating local storey displacements, the deformation due to shear type of mechanisms was not properly determined. Hence, the displacement capacity was underestimated.
- As mentioned previously, shown in Figure 7- 14, linear interpolation was applied in order to plot the shear mechanisms on the pushover curve. As a result, the base shears at the displacement ranges where interpolations were applied were overestimated.

- Though the displaced-shapes (i.e. in terms of ratios) were calculated from local storey displacements, less error might exist compared to applying the local displacements directly in the determination of global displacements. Therefore, it was expected that the approach of using the displaced-shapes might give more accurate results. However, this should be verified by numerical modelling, see Chapter 8.

It is also worth noting that the approach with the "yield state" assumption tends to overestimate the displacement capacity, for the reason that the assumed linear profile under "yield state" was hypothetical, and was overestimated. As a consequence, with the successive addition of displacement from the lateral loading after the "yield state", the ultimate displacement was overestimated.

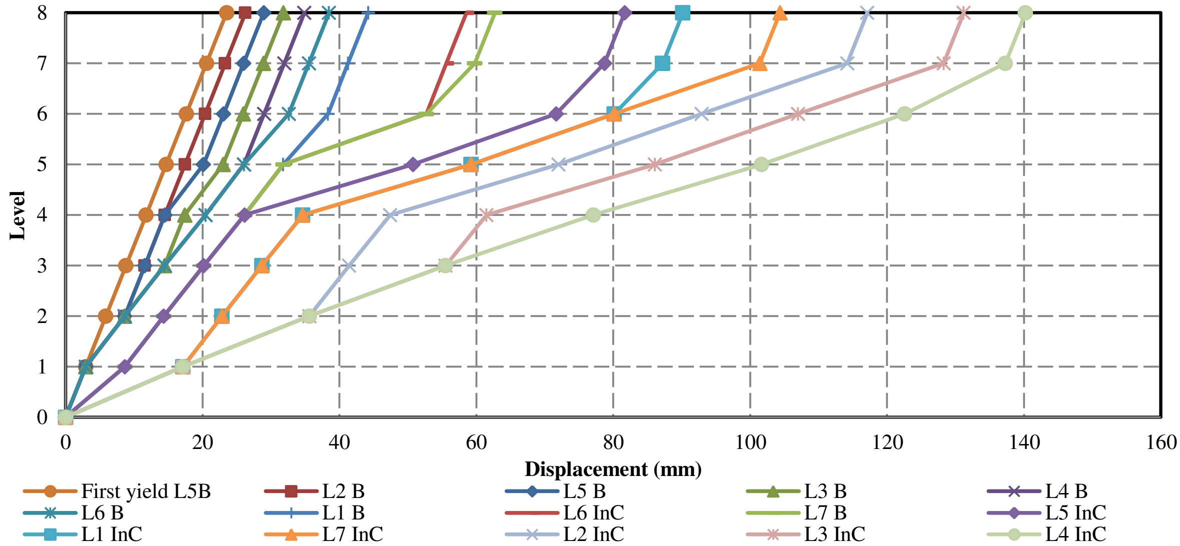


Figure 7- 12: Displaced shapes computed following the sequence of mechanisms determined from Portal Frame Method

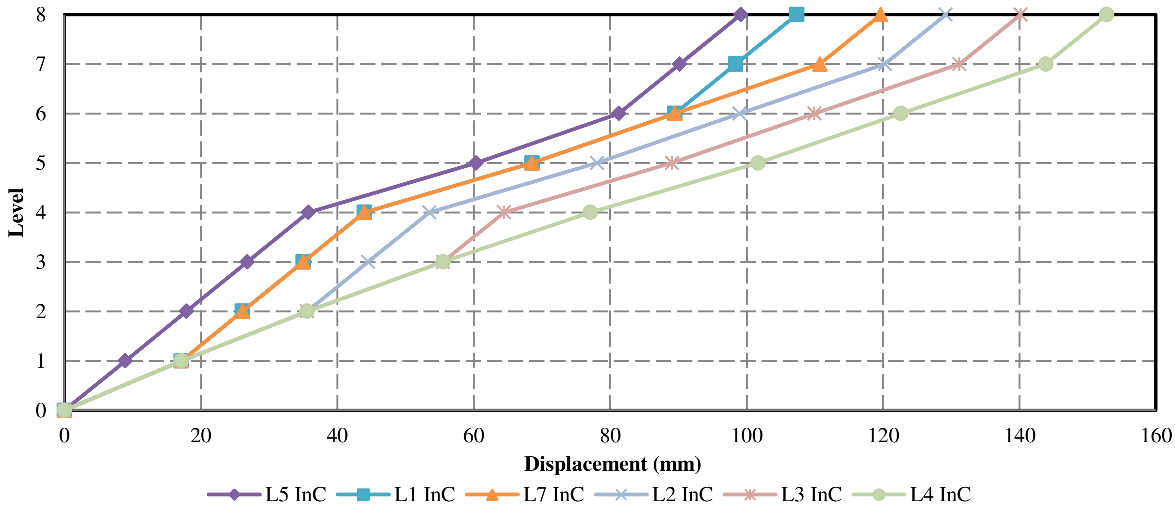


Figure 7- 13: Displaced shapes computed following the sequence of mechanisms determined from Portal Frame Method with a generalised "Yield State" assumption

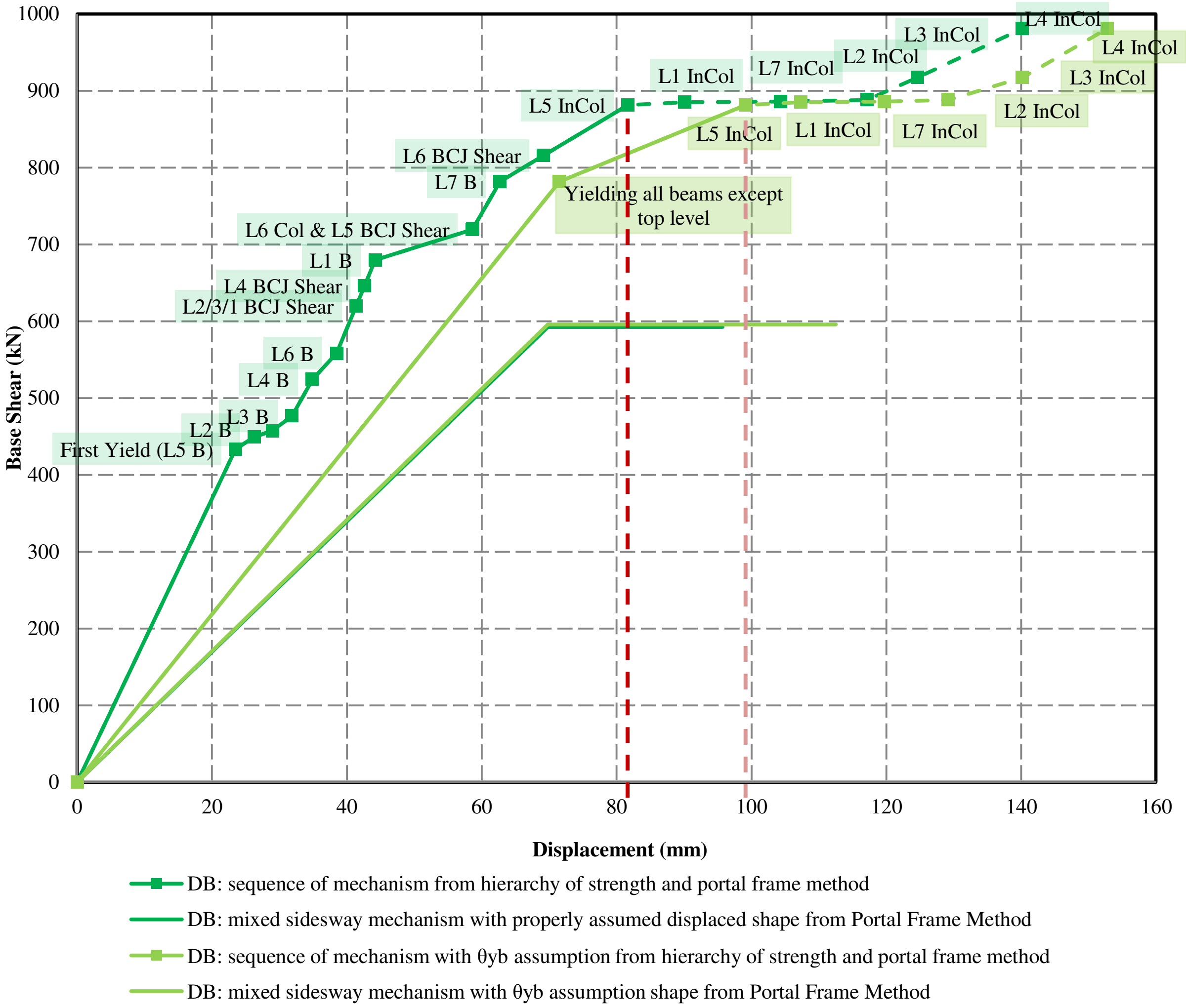


Figure 7- 14: Building No.21 Frame 1 pushover curve with sequence of mechanisms by the improved SLaMa with Evaluation of Strength Hierarchy and Portal Frame Method

7.3.5.4. Adopt Component Analysis Model and Global Structure Model

Since the current New Zealand guidelines do not include any specifications regarding the component analysis model nor global structure model, the first step of applying this approach is to generate such models. Due to timeframe constraint and scope of thesis, only the procedures are discussed without computing the models, see Section 5.3.3.

7.3.5.5. Comparison among the Computed Pushover Curves

From Figure 7- 15 with pushover curves from different approaches shown, the following conclusions can be drawn:

- The findings confirm the limitations of the current SLaMa. The current SLaMa tends to overestimate base shear capacity and displacement capacity.
- The pushover curves from Portal Frame Method (including the two bi-linear simplified curves by applying displaced-shapes calculated by Portal Frame Method) lie between the determined

lower and upper bounds. It is confirmed that the structure was expected to subject to a mixed failure mechanism but more close to a column sidesway mechanism.

- The analytical results should be verified by numerical modelling, see Chapter 8.

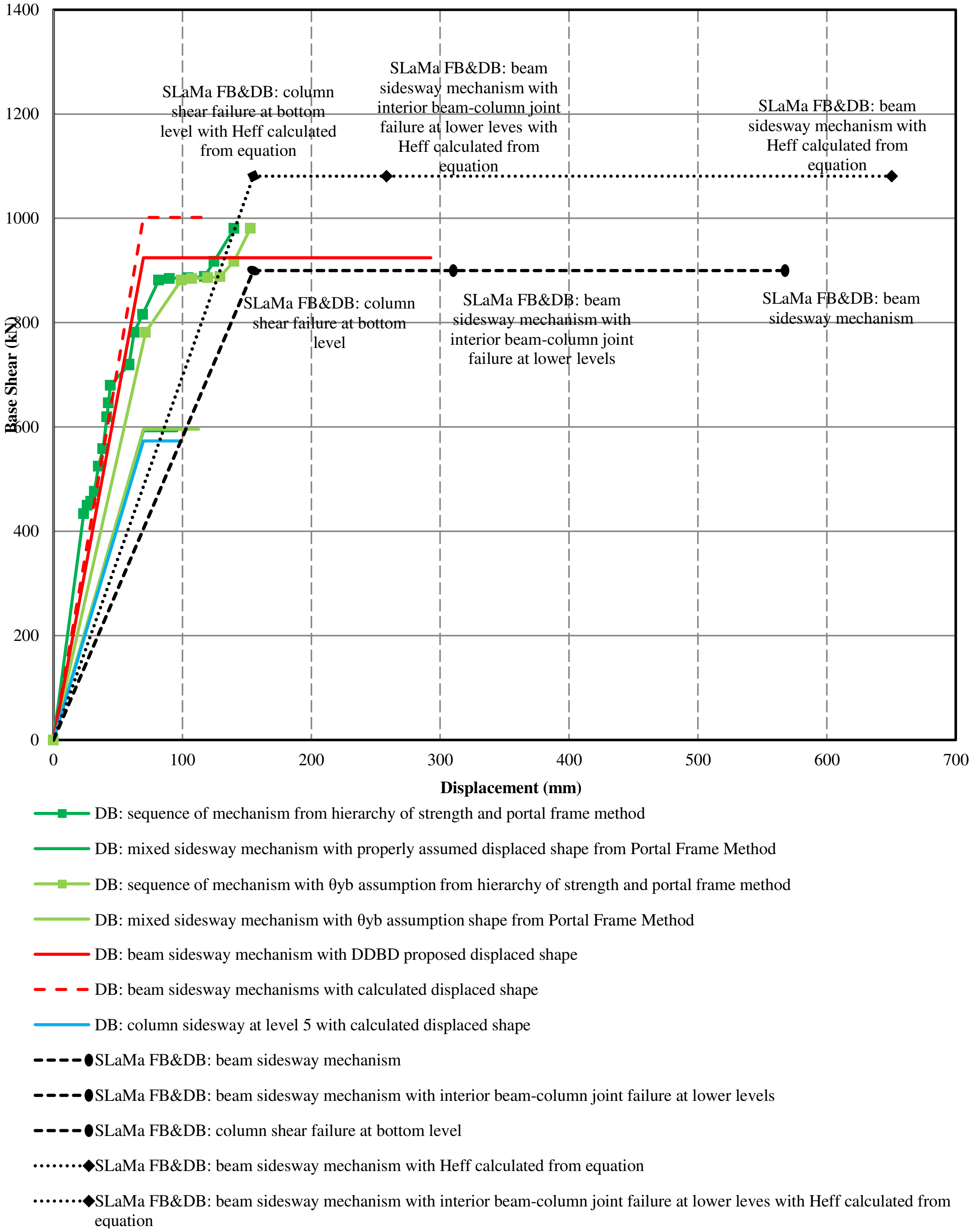


Figure 7- 15: Pushover curves from (1) Current SLaMa (2) Improved SLaMa with Evaluation of Strength Hierarchy and Determination of Lower and Upper Bounds of Lateral Load Capacity (3) Improved SLaMa with Evaluation of Strength Hierarchy and Portal Frame Method

7.3.5.6. Comparison of Assessment Results and the Observed Damages

Frame A and D were also assessed by the current and the improved SLaMa approaches. Since it has been found that similar results were computed, the details are not shown in thesis.

The details of observed damages are shown in Section 7.1.4. From comparison between the assessment results and the observed damages, the following findings were obtained.

- Beam flexural hinging was predicted in the assessment, by both the current and the improved assessment procedures.
- The severe short column failure at level 1 was not predicted in the assessment, for the reason that the interaction between the columns and the deep nonstructural components were not considered in the assessment.
- Joint shear failure was properly predicted by the improved assessment procedures.
- The column sidesway mechanism at level 5 predicted by the improved assessment procedures was not observed in the structure. More investigation should be done regarding comparing the capacity to the seismic demand (Section 9.3).

7.4. Discussion

7.4.1. Influence of Material Strength Variation

As discussed in previous sections (Section 7.3.1 and Section 7.3.2), the nominal material strengths were applied, and the capacities given in Section 7.3.5 were computed as the lower bounds. In this section, the calculation results based on the probable material strengths shown in Table 7- 13 are presented, and the impacts on assessment results due to the variation of material strength are discussed.

7.4.1.1. Component Flexural Capacity

*Table 7- 29: Comparison of beams flexural capacities calculated with nominal material strengths and probable material strengths (**without consideration of axial load on columns**) of Frame 1 in Building No.21*

Level	Beams (nominal material strengths)				Beams (probable material strengths)				Ratio of Change			
	Exterior		Interior		Exterior		Interior		Exterior		Interior	
	ϕy	My	ϕy	My	ϕy	My	ϕy	My	ϕy	My	ϕy	My
	ϕu	Mu	ϕu	Mu	ϕu	Mu	ϕu	Mu	ϕu	Mu	ϕu	Mu
1	0.002809	323.94	0.002809	323.94	0.002984	350.58	0.002984	350.58	6.23%	8.22%	6.23%	8.22%
	0.054114	343.45	0.054114	343.45	0.05696	373.04	0.05696	373.04	5.26%	8.62%	5.26%	8.62%
2	0.002889	289.24	0.002679	214.24	0.003062	312.99	0.002855	231.66	5.99%	8.21%	6.57%	8.13%
	0.05715	301.48	0.067672	220.76	0.060383	326.92	0.070315	239.68	5.66%	8.44%	3.91%	8.57%
3	0.002889	289.24	0.002679	214.24	0.003062	312.99	0.002855	231.66	5.99%	8.21%	6.57%	8.13%
	0.05715	301.48	0.067672	220.76	0.060383	326.92	0.070315	239.68	5.66%	8.44%	3.91%	8.57%
4	0.002889	289.24	0.002679	214.24	0.003062	312.99	0.002855	231.66	5.99%	8.21%	6.57%	8.13%
	0.05715	301.48	0.067672	220.76	0.060383	326.92	0.070315	239.68	5.66%	8.44%	3.91%	8.57%
5	0.00259	160.85	0.002634	160.18	0.002757	174.17	0.002798	173.58	6.45%	8.28%	6.23%	8.37%
	0.054979	166.96	0.053515	166.98	0.059081	182.43	0.059089	182.43	7.46%	9.27%	10.42%	9.25%
6	0.002679	158.11	0.002737	156.79	0.002846	171.25	0.002899	170.04	6.23%	8.31%	5.92%	8.45%
	0.06593	164.64	0.058795	164.59	0.07184	179.07	0.068288	179.09	8.96%	8.76%	16.15%	8.81%
7	0.002679	158.11	0.002737	156.79	0.002846	171.25	0.002899	170.04	6.23%	8.31%	5.92%	8.45%
	0.06593	164.64	0.058795	164.59	0.07184	179.07	0.068288	179.09	8.96%	8.76%	16.15%	8.81%
R	0.002679	158.11	0.002737	156.79	0.002846	171.25	0.002899	170.04	6.23%	8.31%	5.92%	8.45%
	0.06593	164.64	0.058795	164.59	0.07184	179.07	0.068288	179.09	8.96%	8.76%	16.15%	8.81%

(**unit: rad/m for curvature and kNm for moment*)

*Table 7- 30: Comparison of columns flexural capacities calculated with nominal material strengths and probable material strengths (**without consideration of axial load on columns**) of Frame 1 in Building No.21*

Level	Column (nominal material strengths)				Column (probable material strengths)				Ratio of Change			
	Exterior		Interior		Exterior		Interior		Exterior		Interior	
	ϕy	My	ϕy	My	ϕy	My	ϕy	My	ϕy	My	ϕy	My
	ϕu	Mu	ϕu	Mu	ϕu	Mu	ϕu	Mu	ϕu	Mu	ϕu	Mu
1	0.005104	300.16	0.005104	245.53	0.005399	326.85	0.005399	269.11	5.78%	8.89%	5.78%	9.60%
	0.035205	389.48	0.025514	348.91	0.044001	426.7	0.0309307	395.14	24.99%	9.56%	21.23%	13.25%
2	0.005104	300.16	0.005104	245.53	0.005399	326.89	0.005399	269.11	5.78%	8.91%	5.78%	9.60%
	0.035205	389.48	0.025514	348.91	0.044001	426.7	0.030931	395.14	24.99%	9.56%	21.23%	13.25%
3	0.005104	300.16	0.005104	245.53	0.005399	326.89	0.005399	269.11	5.78%	8.91%	5.78%	9.60%
	0.035205	389.48	0.025514	348.91	0.044001	426.7	0.030931	395.14	24.99%	9.56%	21.23%	13.25%
4	0.005104	300.16	0.005104	245.53	0.005399	326.89	0.005399	269.11	5.78%	8.91%	5.78%	9.60%
	0.035205	389.48	0.025514	348.91	0.044001	426.7	0.030931	395.14	24.99%	9.56%	21.23%	13.25%
5	0.004411	130.09	0.004822	177.51	0.004683	141.05	0.005105	194.23	6.17%	8.42%	5.87%	9.42%
	0.076054	136.11	0.035205	260.08	0.084753	149.51	0.045495	287.45	11.44%	9.84%	29.23%	10.52%
6	0.004411	130.09	0.004411	130.09	0.004683	141.05	0.004683	141.05	6.17%	8.42%	6.17%	8.42%
	0.076054	136.11	0.076054	136.11	0.084754	149.51	0.084754	149.51	11.44%	9.84%	11.44%	9.84%
7	0.004411	130.09	0.004411	130.09	0.004683	141.05	0.004683	141.05	6.17%	8.42%	6.17%	8.42%
	0.076054	136.11	0.076054	136.11	0.084754	149.51	0.084754	149.51	11.44%	9.84%	11.44%	9.84%
R	0.004411	130.09	0.004411	130.09	0.004683	141.05	0.004683	141.05	6.17%	8.42%	6.17%	8.42%
	0.076054	136.11	0.076054	136.11	0.084754	149.51	0.084754	149.51	11.44%	9.84%	11.44%	9.84%

*Table 7- 31: Comparison of columns flexural capacities calculated with nominal material strengths and probable material strengths (**with consideration of axial load on columns**) of Frame 1 in Building No.21*

Level	Column (nominal material strengths)				Column (probable material strengths)				Ratio of Change			
	Exterior		Interior		Exterior		Interior		Exterior		Interior	
	ϕy	My	ϕy	My	ϕy	My	ϕy	My	ϕy	My	ϕy	My
	ϕu	Mu	ϕu	Mu	ϕu	Mu	ϕu	Mu	ϕu	Mu	ϕu	Mu
1	0.006552	487.98	0.006296	386.78	0.006738	518.32	0.006502	414.19	2.84%	6.22%	3.27%	7.09%
	0.015441	513.58	0.016845	447.19	0.020249	590.85	0.020678	513.48	31.14%	15.05%	22.75%	14.82%
2	0.006336	459.33	0.006156	369.86	0.006538	489.06	0.006373	396.77	3.19%	6.47%	3.53%	7.28%
	0.017039	499.11	0.017745	436.3	0.021984	571.33	0.021575	500.66	29.02%	14.47%	21.58%	14.75%
3	0.006144	434.05	0.006015	352.73	0.006361	463.24	0.006241	379.15	3.53%	6.73%	3.76%	7.49%
	0.018617	484.89	0.018698	424.87	0.023654	553.57	0.022517	487.54	27.06%	14.16%	20.42%	14.75%
4	0.005948	408.39	0.00587	335.39	0.00618	437.05	0.006108	361.32	3.90%	7.02%	4.05%	7.73%
	0.020386	469.45	0.019706	412.96	0.025484	535.27	0.023503	474.14	25.01%	14.02%	19.27%	14.81%
5	0.005378	227.81	0.005545	254.88	0.005575	240.41	0.005773	273.51	3.66%	5.53%	4.11%	7.31%
	0.041196	244.14	0.022223	321.66	0.051712	259.66	0.028234	372.82	25.53%	6.36%	27.05%	15.90%
6	0.005143	202.85	0.005106	198.93	0.005359	214.93	0.005324	210.93	4.20%	5.96%	4.27%	6.03%
	0.049102	216.37	0.05036	211.96	0.059046	230.84	0.060278	226.33	20.25%	6.69%	19.69%	6.78%
7	0.004914	179.06	0.004886	176.17	0.005147	190.7	0.005121	187.76	4.74%	6.50%	4.81%	6.58%
	0.057126	189.69	0.058161	186.47	0.066824	203.65	0.067816	200.38	16.98%	7.36%	16.60%	7.46%
R	0.004664	154.08	0.004643	152.04	0.004917	165.32	0.004897	163.26	5.42%	7.29%	5.47%	7.38%
	0.06645	162.03	0.067246	159.8	0.075701	175.65	0.076453	173.4	13.92%	8.41%	13.69%	8.51%

As shown in Table 7- 29, Table 7- 30, and Table 7- 31, flexural capacities of beams and columns from Frame 1 of Building No.21 calculated based on nominal material strengths and probable material strengths are presented, with or without the consideration of axial load on columns. The bilinear moment-curvature curves for the beams and columns are shown in the following figures (Figure 7- 16). It has been found that compared to the flexural moments calculated with the application of nominal material strengths (Figure 7- 8), yielding and ultimate moments of all sections calculated with the application of probable material strengths were found to be 2% ~ 10% larger without the consideration of axial load, and 6% ~ 15% larger with the consideration of axial load (for column sections). This is consistent with anticipation that the beams and columns would be "stiffer" (i.e. higher strength) with higher material strengths. It was found that beam curvatures and column yielding curvatures are subjected to generally less significant increases of 5% ~ 9% , while column ultimate curvatures are subjected to more significant increases of 11% ~ 31%

It is worth noting that according to NZSEE 2006, shown in Table 3- 34 (Section 3.3.4.1) and Table 3- 37 (Section 3.3.4.2) and Section 7.3.2, the probable component strengths are estimated using the provided formula $M_{probable}=1.08M_{nominal}$ ($M_{nominal}$ calculated based on nominal material strengths). In Table 7- 32 and Table 7- 33, component probable strengths approximated according to the provided formula and component strengths calculated with the application of probable material strengths are presented. It was found the two procedures generate 0% ~7% differences. It can be suggested that the factor of 1.08 should be well explained in the assessment guidelines in order to give better approximation of component probable strengths.

Table 7- 32: Estimation of beam probable strength

Level	Beam (probable material strengths)		Beam (from NZSEE2006 specification)		Ratio of Change (%)	
	Exterior	Interior	Exterior	Interior	Exterior	Interior
	My	My	My	My	My	My
	Mu	Mu	Mu	Mu	Mu	Mu
1	350.58	350.58	349.86	349.86	0.21%	0.21%
	373.04	373.04	370.93	370.93	0.57%	0.57%
2	312.99	231.66	312.38	231.38	0.20%	0.12%
	326.92	239.68	325.60	238.42	0.41%	0.53%
3	312.99	231.66	312.38	231.38	0.20%	0.12%
	326.92	239.68	325.60	238.42	0.41%	0.53%
4	312.99	231.66	312.38	231.38	0.20%	0.12%
	326.92	239.68	325.60	238.42	0.41%	0.53%
5	174.17	173.58	173.72	172.99	0.26%	0.34%
	182.43	182.43	180.32	180.34	1.17%	1.16%
6	171.25	170.04	170.76	169.33	0.29%	0.42%
	179.07	179.09	177.81	177.76	0.71%	0.75%
7	171.25	170.04	170.76	169.33	0.29%	0.42%
	179.07	179.09	177.81	177.76	0.71%	0.75%
R	171.25	170.04	170.76	169.33	0.29%	0.42%
	179.07	179.09	177.81	177.76	0.71%	0.75%

*Table 7- 33: Column probable strength (**without consideration of axial load on columns**)*

Level	Without consideration of axial load						With consideration of axial load					
	Column (probable material strengths)		Column (from NZSEE2006 specification)		Ratio of Change		Column (probable material strengths)		Column(from NZSEE2006 specification)		Ratio of Change	
	Exterior	Interior	Exterior	Interior	Exterior	Interior	Exterior	Interior	Exterior	Interior	Exterior	Interior
	My	My	My	My	My	My	My	My	My	My	My	My
	Mu	Mu	Mu	Mu	Mu	Mu	Mu	Mu	Mu	Mu	Mu	Mu
1	326.85	269.11	324.17	265.17	0.83%	1.48%	518.32	414.19	527.02	417.72	-1.65%	-0.85%
	426.7	395.14	420.64	376.82	1.44%	4.86%	590.85	513.48	554.67	482.97	6.52%	6.32%
2	326.89	269.11	324.17	265.17	0.84%	1.48%	489.06	396.77	496.08	399.45	-1.41%	-0.67%
	426.7	395.14	420.64	376.82	1.44%	4.86%	571.33	500.66	539.04	471.20	5.99%	6.25%
3	326.89	269.11	324.17	265.17	0.84%	1.48%	463.24	379.15	468.77	380.95	-1.18%	-0.47%
	426.7	395.14	420.64	376.82	1.44%	4.86%	553.57	487.54	523.68	458.86	5.71%	6.25%
4	326.89	269.11	324.17	265.17	0.84%	1.48%	437.05	361.32	441.06	362.22	-0.91%	-0.25%
	426.7	395.14	420.64	376.82	1.44%	4.86%	535.27	474.14	507.01	446.00	5.57%	6.31%
5	141.05	194.23	140.50	191.71	0.39%	1.31%	240.41	273.51	246.03	275.27	-2.29%	-0.64%
	149.51	287.45	147.00	280.89	1.71%	2.34%	259.66	372.82	263.67	347.39	-1.52%	7.32%
6	141.05	141.05	140.50	140.50	0.39%	0.39%	214.93	210.93	219.08	214.84	-1.89%	-1.82%
	149.51	149.51	147.00	147.00	1.71%	1.71%	230.84	226.33	233.68	228.92	-1.22%	-1.13%
7	141.05	141.05	140.50	140.50	0.39%	0.39%	190.7	187.76	193.38	190.26	-1.39%	-1.32%
	149.51	149.51	147.00	147.00	1.71%	1.71%	203.65	200.38	204.87	201.39	-0.59%	-0.50%
R	141.05	141.05	140.50	140.50	0.39%	0.39%	165.32	163.26	166.41	164.20	-0.65%	-0.57%
	149.51	149.51	147.00	147.00	1.71%	1.71%	175.65	173.4	174.99	172.58	0.38%	0.47%

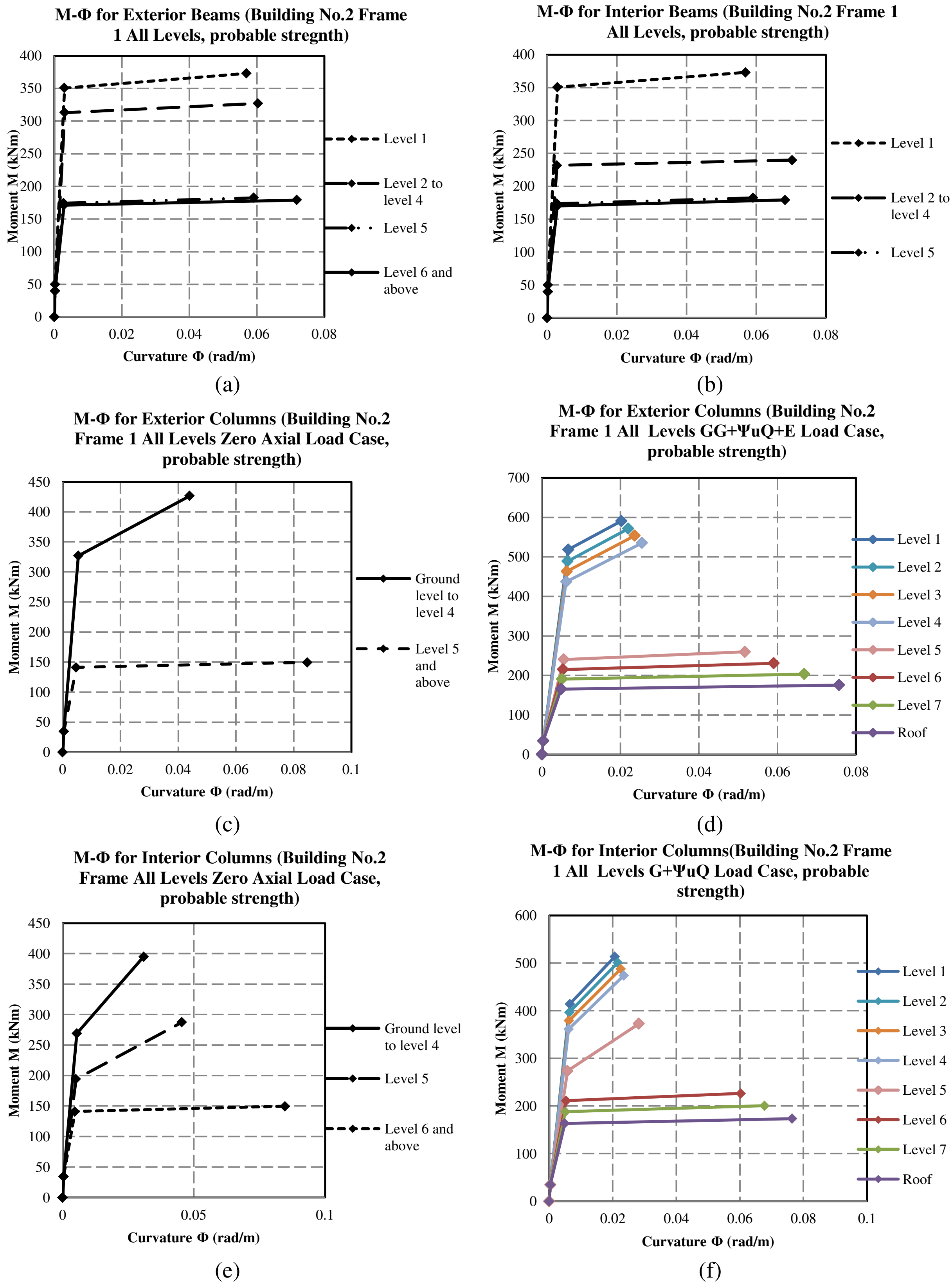


Figure 7- 16: Building No. 21 Frame 1 all levels applying probable strength: (a) Moment-curvature relationship for exterior beams; (b) Moment-curvature relationship for interior beams; (c) Moment-curvature relationship for exterior columns under zero axial load case (i.e. assumed to be the same for the G+ΨuQ-E load case); (d) Moment-curvature relationship for exterior columns under G+ΨuQ+E load case; (e) Moment-curvature relationship for interior columns under zero load case; (f) Moment-curvature relationship for interior columns under G+ΨuQ load case

7.4.1.2. Component Shear Capacity and Demand (at Flexural Capacity)

Table 7- 34: Comparison of exterior beam shear capacities calculated with nominal material strengths and probable material strengths

Level	VBPI k=0.2 (kN)	VBPI k=0.05 (kN)	VBD (kN)	VBPI k=0.2 (kN)	VBPI k=0.05 (kN)	VBD (kN)	Ratio of Change		
R (8)	258.4	171.1	64.7	295.92	189.03	68.98	14.52%	10.48%	6.62%
7	258.4	171.1	64.7	295.92	189.03	68.98	14.52%	10.48%	6.62%
6	258.4	171.1	64.7	295.92	189.03	68.98	14.52%	10.48%	6.62%
5	258.4	171.1	65.3	295.92	189.03	70	14.52%	10.48%	7.20%
4	258.4	171.1	106.6	295.92	189.03	114.01	14.52%	10.48%	6.95%
3	258.4	171.1	106.6	295.92	189.03	114.01	14.52%	10.48%	6.95%
2	258.4	171.1	106.6	295.92	189.03	114.01	14.52%	10.48%	6.95%
1	287.5	178.4	118.9	331.56	197.93	128.06	15.33%	10.95%	7.70%

Table 7- 35: Comparison of interior beam shear capacities calculated with nominal material strengths and probable material strengths

Level	VBPI k=0.2 (kN)	VBPI k=0.05 (kN)	VBD (kN)	VBPI k=0.2 (kN)	VBPI k=0.05 (kN)	VBD (kN)	Ratio of Change		
R (8)	258.4	171.1	54	295.92	189.03	70.45	14.52%	10.48%	30.46%
7	258.4	171.1	54	295.92	189.03	70.45	14.52%	10.48%	30.46%
6	258.4	171.1	54	295.92	189.03	70.45	14.52%	10.48%	30.46%
5	258.4	171.1	66.3	295.92	189.03	71.51	14.52%	10.48%	7.86%
4	258.4	171.1	83.7	295.92	189.03	89.57	14.52%	10.48%	7.01%
3	258.4	171.1	83.7	295.92	189.03	89.57	14.52%	10.48%	7.01%
2	258.4	171.1	83.7	295.9246	189.03	89.57	14.52%	10.48%	7.01%
1	287.5	178.4	122.2	331.56	197.93	131.66	15.33%	10.95%	7.74%

Table 7- 36: Comparison of exterior column shear capacities calculated with nominal material strengths and probable material strengths

														Ratio of Change (%)	
Level	Vc (kN)	Vs (kN)	VCPI (kN)	VCPI (kN) (k=0.1)	VCD (kN)	VCD no ωv (kN)	Vc (kN)	Vs (kN)	VCPI (kN)	VCPI (kN) (k=0.1)	VCD (kN)	VCD no ωv (kN)	VCPI (kN)	VCD (kN)
R (8)	175	138	225	158	63	54	215	149	262	179	69	59	16.44%	9.52%
7	175	138	225	158	63	54	215	149	262	179	69	59	16.44%	9.52%
6	175	138	225	158	64	55	215	149	262	179	69	59	16.44%	7.81%
5	175	138	225	158	90	77	215	149	262	179	97	84	16.44%	7.78%
4	238	206	320	208	115	99	292	223	371	233	125	107	15.94%	8.70%
3	238	206	320	208	115	99	292	223	371	233	125	107	15.94%	8.70%
2	238	206	320	208	123	106	292	223	371	233	134	115	15.94%	8.94%
1	238	206	320	208	393	337	292	223	371	233	452	388	15.94%	15.01%

Table 7- 37: Comparison of interior column shear capacities calculated with nominal material strengths and probable material strengths

													Ratio of Change (%)	
Level	Vc (kN)	Vs (kN)	VCPI (kN)	VCPI (kN) (k=0.1)	VCD (kN)	VCD no ωv (kN)	Vc (kN)	Vs (kN)	VCPI (kN)	VCPI (kN) (k=0.1)	VCD (kN)	VCD no ωv (kN)	VCPI (kN)	VCD (kN)
R (8)	175	138	225	158	126	108	215	149	262	179	137	118	16.44%	8.73%
7	175	138	225	158	126	108	215	149	262	179	137	118	16.44%	8.73%
6	175	138	225	158	127	96	215	149	262	179	138	119	16.44%	8.66%
5	231	138	266	158	164	167	283	149	311	179	178	153	16.92%	8.54%
4	238	275	370	257	201	225	292	297	424	286	217	186	14.59%	7.96%
3	238	275	370	257	201	225	292	297	424	286	217	186	14.59%	7.96%
2	238	275	370	257	231	241	292	297	424	286	251	215	14.59%	8.66%
1	238	275	370	257	342	293	292	297	424	286	393	337	14.59%	14.91%

Table 7- 38: Comparison of exterior joint shear capacities calculated with nominal material strengths and probable material strengths

Axial Load	N*(G+ΨQ)		N*(G+ΨQ+E)		N*(G+ΨQ-E)		N*=0		
Level	Vpji k=0.4 (kN)	Vpji k=0.3 (kN)	Vpji k=0.4 (kN)	Vpji k=0.3 (kN)	Vpji k=0.4 (kN)	Vpji k=0.3 (kN)	Vpji k=0.4 (kN)	Vpji k=0.3 (kN)	Vij (kN)
R (8)	378	290	403	314	351	264	349	262	191
7	408	320	453	362	358	271	349	262	191
6	436	346	498	404	363	276	349	262	191
5	462	370	542	445	363	276	349	262	193
4	486	392	585	484	359	272	349	262	348
3	508	413	630	524	346	259	349	262	348
2	530	433	672	562	332	245	349	262	348
1	551	452	717	602	305	217	349	262	397
Axial Load	N*(G+ΨQ)		N*(G+ΨQ+E)		N*(G+ΨQ-E)		N*=0		
Level	Vpji k=0.4 (kN)	Vpji k=0.3 (kN)	Vpji k=0.4 (kN)	Vpji k=0.3 (kN)	Vpji k=0.4 (kN)	Vpji k=0.3 (kN)	Vpji k=0.4 (kN)	Vpji k=0.3 (kN)	Vij (kN)
R (8)	456	349	482	374	430	323	428	321	207
7	488	380	534	424	436	329	428	321	207
6	516	407	581	468	442	335	428	321	207
5	543	432	627	511	441	334	428	321	211
4	568	455	673	553	438	331	428	321	378
3	592	478	721	596	425	318	428	321	378
2	614	499	766	637	411	304	428	321	378
1	636	519	814	680	384	276	428	321	432
Ratio of Change (%)									
R (8)	20.63%	20.34%	19.60%	19.11%	22.51%	22.35%	22.64%	22.52%	8.38%
7	19.61%	18.75%	17.88%	17.13%	21.79%	21.40%	22.64%	22.52%	8.38%
6	18.35%	17.63%	16.67%	15.84%	21.76%	21.38%	22.64%	22.52%	8.38%
5	17.53%	16.76%	15.68%	14.83%	21.49%	21.01%	22.64%	22.52%	9.33%
4	16.87%	16.07%	15.04%	14.26%	22.01%	21.69%	22.64%	22.52%	8.62%
3	16.54%	15.74%	14.44%	13.74%	22.83%	22.78%	22.64%	22.52%	8.62%
2	15.85%	15.24%	13.99%	13.35%	23.80%	24.08%	22.64%	22.52%	8.62%
1	15.43%	14.82%	13.53%	12.96%	25.90%	27.19%	22.64%	22.52%	8.82%

Table 7- 39: Comparison of interior joint shear capacities calculated with nominal material strengths and probable material strengths

Axial Load	N*(G+ΨQ)		Vij (kN)	N*(G+ΨQ)		Vij (kN)	Ratio of Change (%)		
Level	Vpji k=1.0 (kN)	Vpji k=0.3 (kN)		Vpji k=1.0 (kN)	Vpji k=0.3 (kN)				
R (8)	924	310	382	1120	370	414	21.21%	19.35%	8.38%
7	979	357	382	1176	418	414	20.12%	17.09%	8.38%
6	1029	397	382	1228	461	414	19.34%	16.12%	8.38%
5	1077	434	386	1277	499	422	18.57%	14.98%	9.33%
4	1122	467	604	1324	535	655	18.00%	14.56%	8.44%
3	1166	498	604	1369	568	655	17.41%	14.06%	8.44%
2	1208	527	604	1413	599	655	16.97%	13.66%	8.44%
1	1248	554	793	1455	629	863	16.59%	13.54%	8.83%

In Table 7- 34, Table 7- 35, Table 7- 36, Table 7- 37, Table 7- 38, and Table 7- 39, comparisons of component (exterior and interior beams, exterior and interior columns, exterior and interior joints) shear strengths using nominal material strengths and probable material strengths are presented. It was found that in general the component shear strengths calculated based on the probable material strengths are approximately 10% ~ 27% larger than the shear strengths calculated based on the nominal material strengths. It was also found that the shear demands estimated based on the component flexural strengths are 6% ~ 30% larger. In spite of the increase of the shear strengths, it was still expected that columns at level 1 and joints at lower levels are prone to shear failure.

To summarise the findings in Section 7.4.1.1 and Section 7.4.1.2, component flexural and shear capacities were found to be increased due to the use of probable material strengths. It is more vital to investigate how the variation of material strength influences the sequence of mechanisms at subassembly level and at global level, which are discussed in Section 7.4.1.3 and Section 7.4.1.4.

7.4.1.3. Strength Hierarchy at Local Level (i.e. subassembly level)

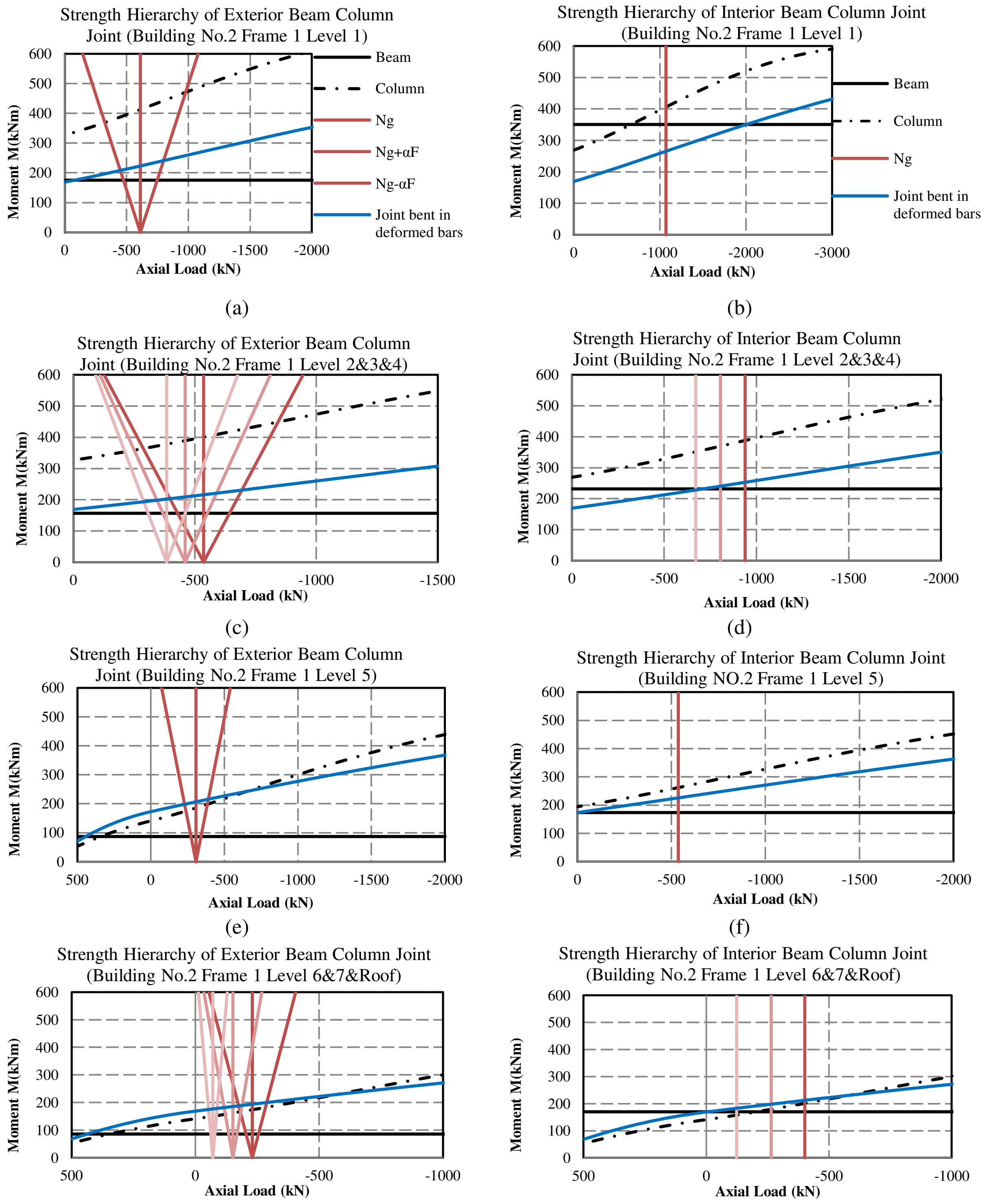


Figure 7- 17: Strength hierarchy evaluation for joints in Frame 1 of Building No.21 applying probable strength: (a) Exterior joints at level 1; (b) Interior joints at level 1; (c) Exterior joints at level 2&3&4; (d) Interior joints at level 2&3&4; (e) Exterior joints at level 5; (f) Interior joints at level 5; (g) Exterior joints at level 6&7&Roof; (h) Interior joints at level 6&7&Roof

By comparing Figure 7- 17 to Figure 7- 9, it can be concluded that:

- In general, same sequences of mechanisms were found at subassembly level.
- When applying probable material strengths, for the exterior joints at all levels and the interior joints at level 1, 2, 5, 6, 7, the strength hierarchy was found to be same with that calculated based on the nominal material strengths.
- When applying probable material strengths, for the interior joints at level 3, 4, roof, the joint strengths were found to be higher than the beam yielding strengths but with only small disparities.

7.4.1.4. Global Mechanisms

In Figure 7- 18, the changes of pushover curves computed by current SLaMa (procedures from NZSEE 2006 Appendix 4E and NZSEE 2006 Section 7) are illustrated. And in Figure 7- 19, the changes of pushover curves computed by the improved SLaMa with Evaluation of Strength Hierarchy and Determination of Lower and Upper Bounds of Lateral Load Capacity are illustrated. Table 7- 40 provides a summary of comparison between the base shear or ultimate displacement capacities calculated using nominal material strengths and probable material strengths, with ratios of differences shown.

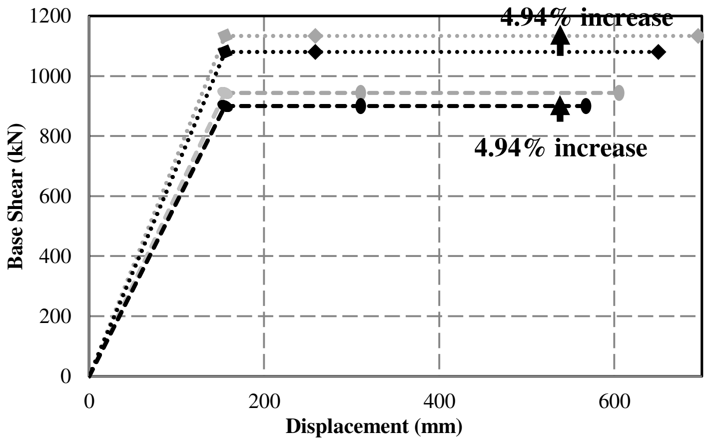


Figure 7- 18: Pushover curves by the current SLaMa method using nominal and probable material strengths

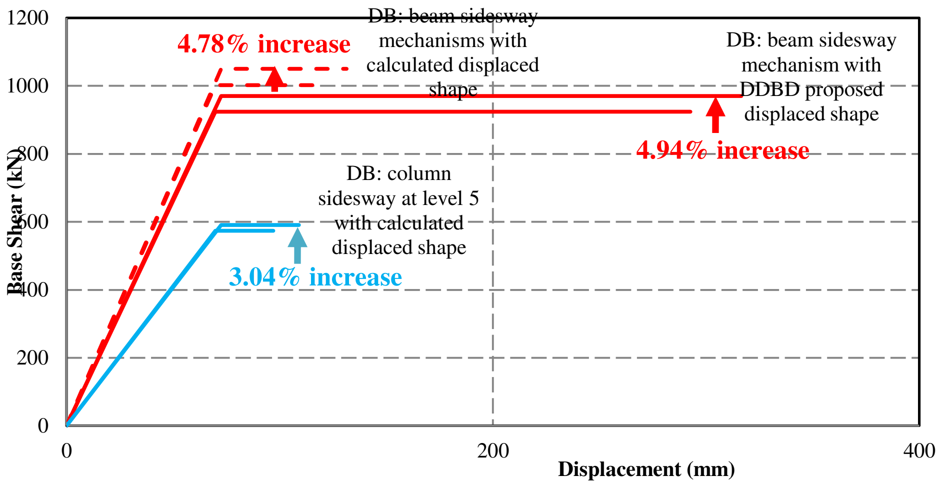


Figure 7- 19: Upper and lower bounds by the improved SLaMa using nominal and probable material strengths

Table 7- 40: Comparison of computed base shear capacity and ultimate displacement capacity by applying nominal material strengths and probable material strengths

Procedures	Base shear (applying nominal material strengths) (kN)	Base shear (applying probable material strengths) (kN)	Ratio of increase	Ultimate displacement (applying nominal material strengths) (mm)	Ultimate displacement (applying probable material strengths) (mm)	Ratio of increase
SLaMa (NZSEE2006 Appendix 4E)	900	945	4.94%	568	605	6.67%
NZSEE2006 Section 7	1081	1134	4.94%	650	696	6.99%
Column sidesway mechanism calculated following Priestley Book procedure	573	591	3.04%	97	110	12.45%
Beam sidesway mechanism calculated following Priestley Book procedure	1002	1050	4.78%	122	131	7.43%
DDBD procedure	924	970	4.94%	293	317	8.15%
Portal Frame Method	593	611	3.03%	96	98	2.84%
Portal Frame Method with yielding state assumption	596	614	3.03%	113	116	2.84%

It is worth noting that the lower bound of lateral load was still estimated by the critical mechanism "column sidesway at level 5", and it was anticipated that the real response of the frame (with mixed sidesway mechanism) should be close to the lower bound.

In Table 7- 41, the comparison of the sequence of mechanisms determined by Portal Frame Method with the application of nominal material strengths and probable material strengths are shown, and the bi-linear pushover curves computed are shown in Figure 7- 20. The differences found are summarised as following:

- In general, similar sequences of mechanisms were computed at global level.
- Joint shear failure mechanisms were found to be delayed when applying probable material strengths. Regarding the joint shear failure mechanisms, the sequence "level 2 – level 3 – level 1 – level 4 – level 5 – level 6 – level 7" changes to "level 2 – level 1 – level 3 – level 4 – level 5 - level 6 – level7".
- Regarding the interior column flexural hinging mechanisms, the sequence "level 5 – level 1 – level 7 – level 2 – level 3 – level 4" changes to "level 7 – level 1 – level 2 – level 5 – level 3 – level 4".
- The results from Portal Frame Method are consistent with the results from the evaluation of strength hierarchy.
- The bi-linear pushover curve computed based on the probable material strengths is similar to the curve computed based on the nominal material strengths, but with larger base shear capacity and ultimate displacement capacity, as shown in Table 7- 42.

Table 7- 41: Comparison of the sequence of mechanisms determined by Portal Frame Method using nominal and probable material strengths

Applying nominal material strength		Applying probable material strength	
Level	**Mechanism**	**Level**	**Mechanism**
Level 5	Beam yield (structure "First Yield" state)	Level 5	Beam yield (structure "First Yield" state)
Level 2	Interior beam flexural hinging	Level 2	Interior beam flexural hinging
Level 5	Exterior beam flexural hinging	Level 5	Exterior beam flexural hinging
Level 5	Interior beam flexural hinging	Level 5	Interior beam flexural hinging
Level 3	Interior beam flexural hinging	Level 3	Interior beam flexural hinging
Level 4	Interior beam flexural hinging	Level 4	Interior beam flexural hinging
Level 6	Beam yield	Level 6	Beam yield
Level 6	Exterior beam flexural hinging	Level 6	Exterior beam flexural hinging
Level 6	Interior beam flexural hinging	Level 6	Interior beam flexural hinging
Level 2	Beam yield	Level 2	Beam yield
Level 2	Exterior beam flexural hinging	Level 2	Exterior beam flexural hinging
Level 2	Joint shear failure	Level 3	Beam yield
Level 3	Beam yield	Level 1	Beam yield
Level 3	Joint shear failure	Level 3	Exterior beam flexural hinging
Level 1	Joint shear failure	Level 2	Joint shear failure
Level 1	Beam yield	Level 1	Joint shear failure
Level 3	Exterior beam flexural hinging	Level 3	Joint shear failure
Level 4	Joint shear failure	Level 1	Exterior beam flexural hinging
Level 1	Exterior beam flexural hinging	Level 4	Beam yield
Level 4	Beam yield	Level 1	Interior beam flexural hinging
Level 1	Interior beam flexural hinging	Level 4	Joint shear failure
Level 4	Exterior beam flexural hinging	Level 4	Exterior beam flexural hinging
Level 6	Interior column flexural hinging	Level 6	Interior column flexural hinging
Level 5	Joint shear failure	Level 7	Beam yield
Level 7	Beam yield	Level 7	Exterior beam flexural hinging
Level 7	Exterior beam flexural hinging	Level 5	Joint shear failure
Level 7	Interior beam flexural hinging	Level 7	Interior beam flexural hinging
Level 6	Joint shear failure	Level 7	Interior column flexural hinging
Level 5	Interior column flexural hinging	Level 6	Joint shear failure
Level 1	Interior column flexural hinging	Level 1	Interior column flexural hinging
Level 7	Interior column flexural hinging	Level 2	Interior column flexural hinging
Level 2	Interior column flexural hinging	Level 5	Interior column flexural hinging
Level 3	Interior column flexural hinging	Level 3	Interior column flexural hinging
Level 4	Interior column flexural hinging	Level 4	Interior column flexural hinging
Level 7	Joint shear failure	Level 7	Joint shear failure
Level 5	Exterior column flexural hinging	Level 5	Exterior column flexural hinging
Level 5	Column shear failure	Level R	Beam yield
Level 1	Column shear failure	Level 6	Exterior column flexural hinging
Level R	Beam yield	Level R	Interior column flexural hinging
Level 2	Column shear failure	Level R	Exterior beam flexural hinging
Level R	Interior column flexural hinging	Level 5	Column shear failure
Level R	Exterior beam flexural hinging	Level 1	Column shear failure
Level 6	Exterior column flexural hinging	Level R	Interior beam flexural hinging
Level R	Interior beam flexural hinging	Level 2	Column shear failure
Level 3	Column shear failure	Level 3	Column shear failure
Level 6	Column shear failure	Level 6	Column shear failure
Level 4	Column shear failure	Level 7	Exterior column flexural hinging
Level 7	Exterior column flexural hinging	Level 4	Column shear failure
Level R	Joint shear joint	Level R	Joint shear joint
Level 2	Exterior column flexural hinging	Level 2	Exterior column flexural hinging
Level 1	Exterior column flexural hinging	Level 1	Exterior column flexural hinging
Level 3	Exterior column flexural hinging	Level 3	Exterior column flexural hinging
Level 7	Column shear failure	Level 4	Exterior column flexural hinging
Level 4	Exterior column flexural hinging	Level 7	Column shear failure
Level R	Exterior column flexural hinging	Level R	Exterior column flexural hinging
Level R	Column shear failure	Level R	Column shear failure

Table 7- 42: Summary of base shears and ultimate displacements calculated by the improved SLaMa with Portal Frame Method using nominal and probable material strengths

Capacity	Applying nominal material strengths	Applying probable material strengths	Ratio of Increase
Base shear (kN)	593	611	3.03%
Ultimate displacement (mm)	96	98	2.84%

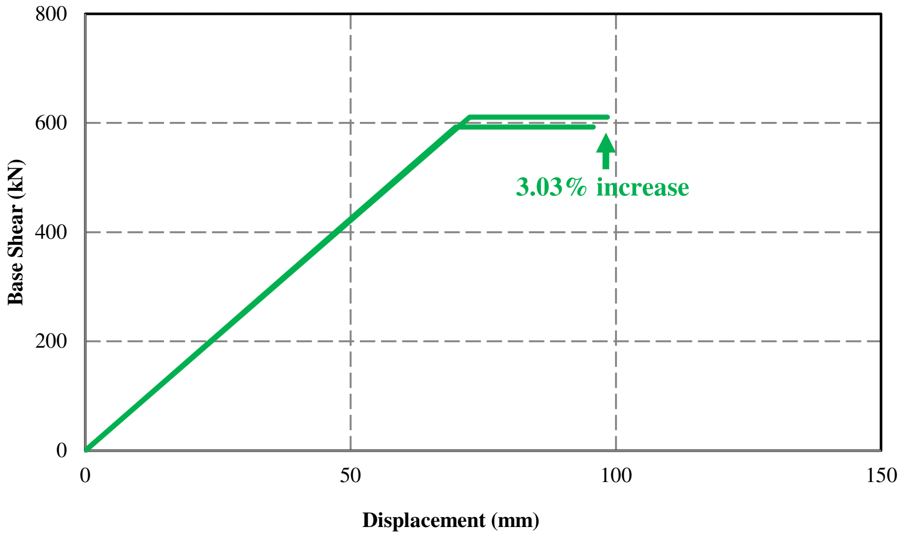


Figure 7- 20: Pushover curves by the improved SLaMa with Portal Frame Method using nominal and probable material strengths

7.4.2. Influence of Component Strength Variation

As discussed in previous section (Section 7.3.2), the nominal component strengths were applied, computing a lower bound of lateral load capacity. In Section 7.4.1, the probable material strengths were applied (approximated probable component strengths), and the results indicate that the assessment outcomes are not significantly affected by using probable strengths instead of using nominal strengths:

- Minor changes to the sequence of mechanisms at subassembly level are found.
- Only a few changes to the sequence of mechanisms at global level are found.
- 3% ~ 5% increases (in different procedures) of base shear capacities were found.
- 2% ~ 13% increases (in different procedures) of ultimate displacement capacities were found.

If applying beam and column overstrengths simultaneously, in this particular case (Building No.21 Frame 1), it is likely that the sequence of mechanisms may still remain the same. Therefore, in order to show some significant impacts on assessment results, in this section, the component properties were artificially changed. In Section 7.4.2.1, the influence due to the change of beam section properties are discussed, and in Section 7.4.2.2, the influence due to the change of both beam and column section properties are discussed. It should be noted that the detailed calculation processes are not presented, but only the final results and discussions are shown.

7.4.2.1. Change of Beam Sections

In Table 7- 43, the changes made to the beam sections are illustrated. About 22% ~ 34% of beam flexural strengths were reduced, as shown in Table 7- 44. The computed moment-curvature curves are shown in Figure 7- 21.

Table 7- 43: Original and changed beam section profiles

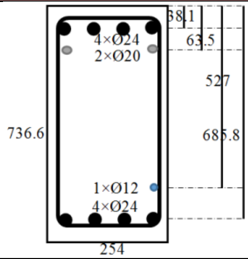
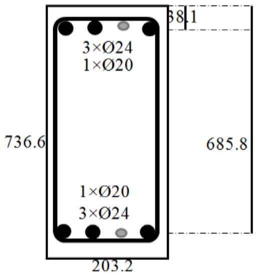
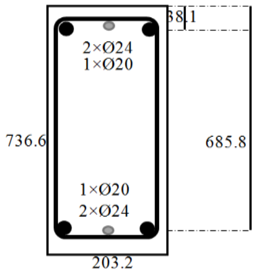
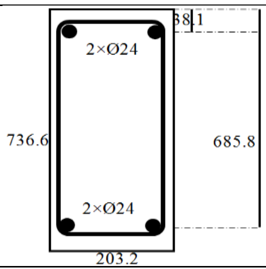
Level	Exterior Beam Sections		Interior Beam Sections
Level 1		**Original**	Same as exterior beams
		Change	Only keep the top and bottom layers: 4×Φ20
Level 2,3,4		**Original**	
		Change	Top and bottom layers 3×Φ20
Level 5,6,7,R		**Original**	Same as exterior beams
		Change	Level 5: keep the same Level 6, 7, R: Top and bottom layers 2×Φ20

Table 7- 44: Comparison of interior beam flexural capacities before and after reduced strength

Level	Original beams		Reduced strength		Ratio of change	
	Interior		Interior		Interior	
	My	My	My	My	My	My
	Mu	Mu	Mu	Mu	Mu	Mu
1	0.002809	323.94	0.002712	219.18	-3.44%	-32.34%
	0.054114	343.45	0.064169	228.25	18.58%	-33.54%
2	0.002679	214.24	0.002692	164.58	0.48%	-23.18%
	0.067672	220.76	0.065234	171.38	-3.60%	-22.37%
3	0.002679	214.24	0.002692	164.58	0.48%	-23.18%
	0.067672	220.76	0.065234	171.38	-3.60%	-22.37%
4	0.002679	214.24	0.002692	164.58	0.48%	-23.18%
	0.067672	220.76	0.065234	171.38	-3.60%	-22.37%
5	0.002634	160.18	0.002634	160.18	0.00%	0.00%
	0.053515	166.98	0.053515	166.98	0.00%	0.00%
6	0.002737	156.79	0.002575	110.60	-5.93%	-29.46%
	0.058795	164.59	0.072970	115.24	24.11%	-29.98%
7	0.002737	156.79	0.002575	110.60	-5.93%	-29.46%
	0.058795	164.59	0.072970	115.24	24.11%	-29.98%
R	0.002737	156.79	0.002575	110.60	-5.93%	-29.46%
	0.058795	164.59	0.072970	115.24	24.11%	-29.98%

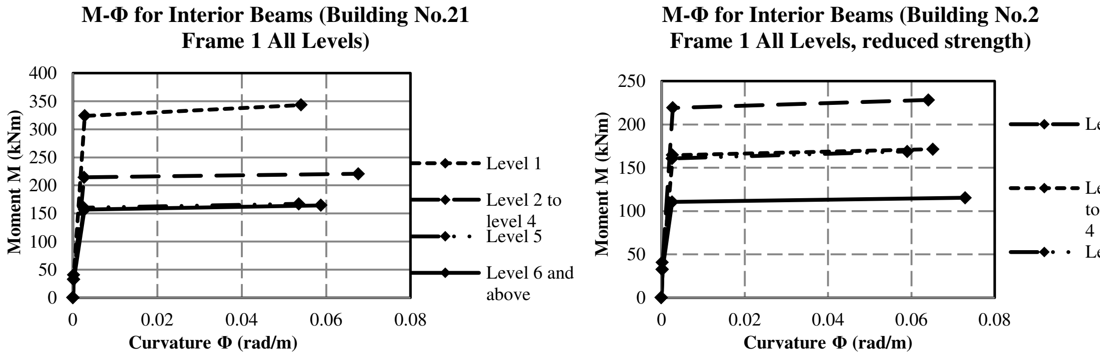


Figure 7- 21: Comparison of moment-curves of interior beams before and after reduced strength

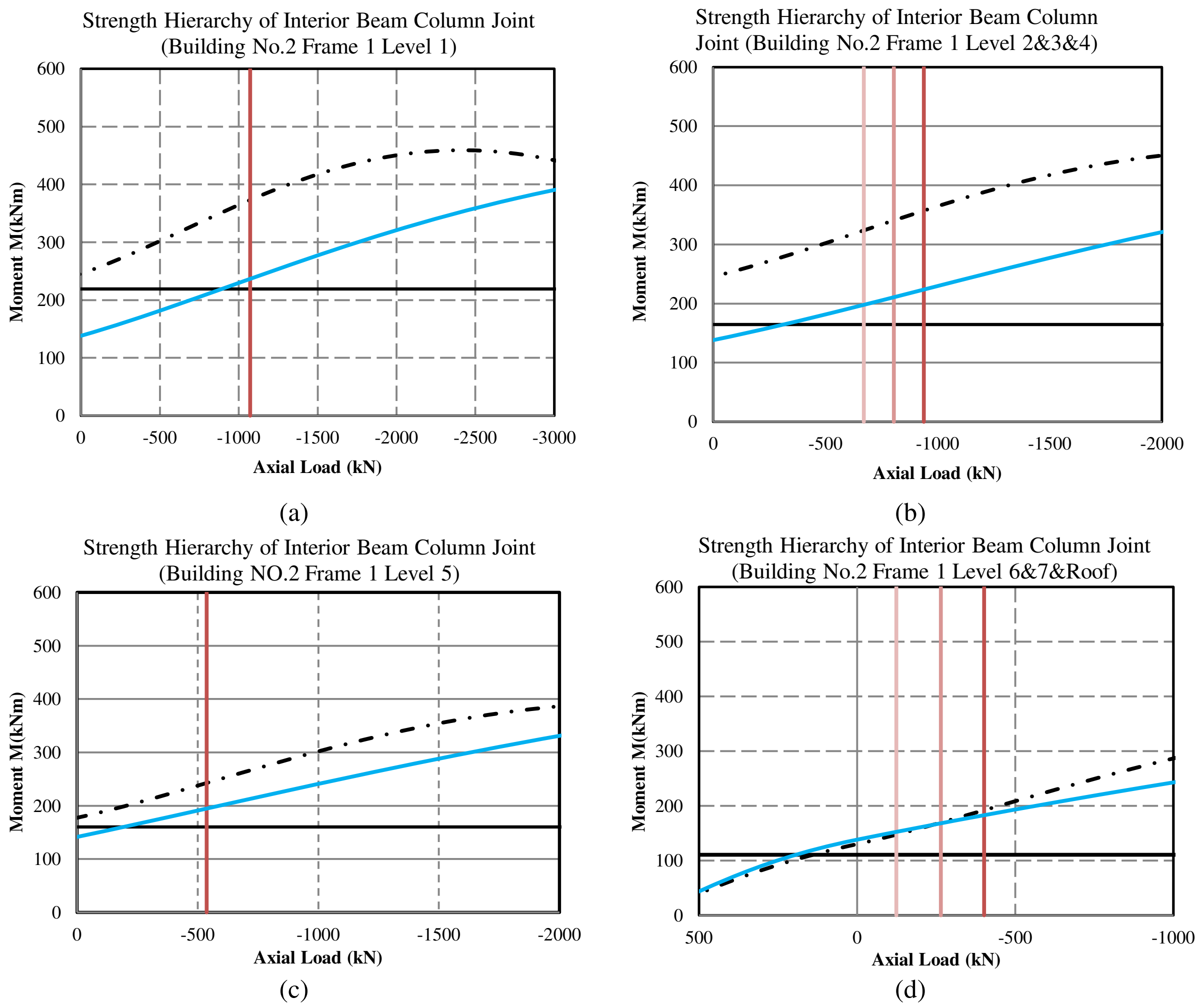


Figure 7- 22: Strength hierarchy evaluation for interior joints in Frame 1 of Building No.21: (a) Interior joints at level 1; (b) Interior joints at level 2, 3, 4; (c) Interior joints at level 5; (d) Interior joints at level 6&7&Roof

As shown in Figure 7- 22, all interior joints follow the sequence “beam flexural hinging – joint shear failure – column flexural hinging”.

Table 7- 45: Sequence of mechanisms determined by Portal Frame method

Component nominal strengths		Reduced beam strengths	
Level	**Mechanism**	**Level**	**Mechanism**
Level 5	Beam yield (structure "First Yield" state)	Level 2	Interior beam flexural hinging
Level 2	Interior beam flexural hinging	Level 3	Interior beam flexural hinging
Level 5	Exterior beam flexural hinging	Level 6	Interior beam flexural hinging
Level 5	Interior beam flexural hinging	Level 4	Interior beam flexural hinging
Level 3	Interior beam flexural hinging	Level 5	Beam yield
Level 4	Interior beam flexural hinging	Level 5	Exterior beam flexural hinging
Level 6	Beam yield	Level 1	Interior beam flexural hinging
Level 6	Exterior beam flexural hinging	Level 5	Interior beam flexural hinging
Level 6	Interior beam flexural hinging	Level 6	Beam yield
Level 2	Beam yield	Level 7	Interior beam flexural hinging
Level 2	Exterior beam flexural hinging	Level 6	Exterior beam flexural hinging
Level 2	Joint shear failure	Level 2	Beam yield flexural hinging
Level 3	Beam yield	Level 2	Exterior beam flexural hinging
Level 3	Joint shear failure	Level 2	Joint shear failure
Level 1	Joint shear failure	Level 3	Beam yield
Level 1	Beam yield	Level 3	Joint shear failure
Level 3	Exterior beam flexural hinging	Level 1	Joint shear failure
Level 4	Joint shear failure	Level 1	Beam yield
Level 1	Exterior beam flexural hinging	Level 3	Exterior beam flexural hinging
Level 4	Beam yield	Level 4	Joint shear failure
Level 1	Interior beam flexural hinging	Level 1	Exterior beam flexural hinging
Level 4	Exterior beam flexural hinging	Level 4	Beam yield
Level 6	Interior column flexural hinging	Level 4	Exterior beam flexural hinging
Level 5	Joint shear failure	Level 6	Interior column flexural hinging
Level 7	Beam yield	Level 5	Joint shear failure
Level 7	Exterior beam flexural hinging	Level 7	Beam yield
Level 7	Interior beam flexural hinging	Level 7	Exterior beam flexural hinging
Level 6	Joint shear failure	Level 6	Joint shear failure
Level 5	Interior column flexural hinging	Level 5	Interior column flexural hinging
Level 1	Interior column flexural hinging	Level 1	Interior column flexural hinging
Level 7	Interior column flexural hinging	Level 7	Interior column flexural hinging
Level 2	Interior column flexural hinging	Level 2	Interior column flexural hinging
Level 3	Interior column flexural hinging	Level 3	Interior column flexural hinging
Level 4	Interior column flexural hinging	Level 4	Interior column flexural hinging
Level 7	Joint shear	Level R	Interior beam flexural hinging
Level 5	Exterior column flexural hinging	Level 7	Joint shear failure
Level 5	Column shear failure	Level 5	Exterior column flexural hinging
Level 1	Column shear failure	Level 5	Column shear failure
Level R	Beam yield	Level 1	Column shear failure
Level 2	Column shear failure	Level R	Beam yield
Level R	Interior column flexural hinging	Level 2	Column shear failure
Level R	Exterior beam flexural hinging	Level R	Interior column flexural hinging
Level 6	Exterior column flexural hinging	Level R	Exterior beam flexural hinging
Level R	Interior beam flexural hinging	Level 6	Exterior column flexural hinging
Level 3	Column shear failure	Level 3	Column shear failure
Level 6	Column shear failure	Level 6	Column shear failure
Level 4	Column shear failure	Level 4	Column shear failure
Level 7	Exterior column flexural hinging	Level 7	Exterior column flexural hinging
Level R	Interior Joint shear joint	Level R	Joint shear failure
Level 2	Exterior column flexural hinging	Level 2	Exterior column flexural hinging
Level 1	Exterior column flexural hinging	Level 1	Exterior column flexural hinging
Level 3	Exterior column flexural hinging	Level 3	Exterior column flexural hinging
Level 7	Column shear failure	Level 7	Column shear failure
Level 4	Exterior column flexural hinging	Level 4	Exterior column flexural hinging
Level R	Exterior column flexural hinging	Level R	Exterior column flexural hinging
Level R	Column shear failure	Level R	Column shear failure

Table 7- 46: Summary of base shears and ultimate displacements calculated by the improved SLaMa with Portal Frame Method using nominal and probable material strengths

Capacity	Original interior beam strengths	Reduced interior beam strengths	Ratio of Increase
Base shear (kN)	593	590	-0.38%
Ultimate displacement (mm)	96	108	12.54%

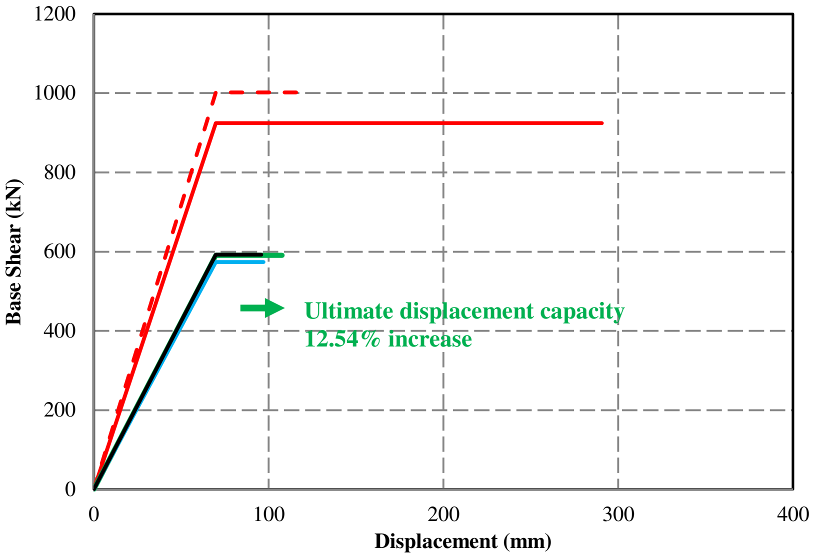


Figure 7- 23: Comparison of the pushover curves computed by the improved SLaMa with Portal Frame Method before and after interior beam strengths reduced

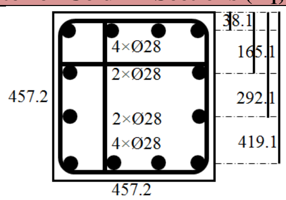
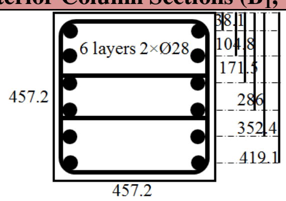
In Table 7- 45, the comparison of the sequence of mechanisms determined by Portal Frame Method with the application of original component strengths and the reduced beam strengths are shown, and the bi-linear pushover curves computed are shown in Figure 7- 23. The differences found are summarised as following:

- In general, similar sequences of mechanisms were computed at global level.
- Joint shear failure mechanisms are found to be delayed when applying the reduced beam strengths.
- As shown in Figure 7- 23 and Table 7- 46, the base shear capacity calculated using the reduced beam strengths was found to be almost the same with the original base shear capacity, with only 0.38% differences. The ultimate displacement capacity calculated using the reduced beam strengths was found to be 12.54% larger than the original ultimate displacement.

7.4.2.2. Change of Both Beam and Column Sections

The changes applied to beams are the same as shown in previous, i.e. Table 7- 43, and the changes made to the column sections are illustrated in Table 7- 47. About 20% ~ 60% of column flexural strengths were increased, as shown in Table 7- 48.

Table 7- 47: Original and changed column section profiles

Level		Exterior Column Sections (A_1, D_1)	Interior Column Sections (B_1, C_1)
Ground level to Level 4 above	Original		
	Change	Same as the oringal	Same as the oringal

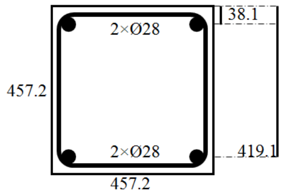
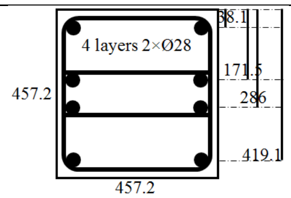
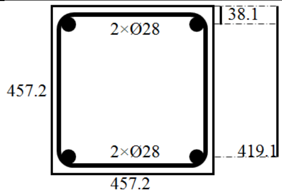
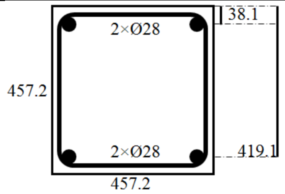
Level 4 above to Level 5 above	Original		
	Change	Top and bottom layers: 3×Φ28 Add two mid layers: 2×Φ28	Top and bottom layers: 3×Φ28
Level 5 above to Roof	Original		
	Change	Top and bottom layers: 3×Φ28	Top and bottom layers: 3×Φ28

Table 7- 48: Comparison of column flexural capacities before and after increased strength at higher levels

Level	Column				Column				Ratio of Change (%)			
	Exterior		Interior		Exterior		Interior		Exterior		Interior	
	My	My	My	My	My	My	My	My	My	My	My	My
	Mu	Mu	Mu	Mu	Mu	Mu	Mu	Mu	Mu	Mu	Mu	Mu
1	0.006552	487.98	0.006296	386.78	0.006552	487.98	0.006296	386.78	0.00%	0.00%	0.00%	0.00%
	0.015441	513.58	0.016845	447.19	0.015441	513.58	0.016845	447.19	0.00%	0.00%	0.00%	0.00%
2	0.006336	459.33	0.006156	369.86	0.006336	459.33	0.006156	369.86	0.00%	0.00%	0.00%	0.00%
	0.017039	499.11	0.017745	436.3	0.017039	499.11	0.017745	436.30	0.00%	0.00%	0.00%	0.00%
3	0.006144	434.05	0.006015	352.73	0.006144	434.05	0.006015	352.73	0.00%	0.00%	0.00%	0.00%
	0.018617	484.89	0.018698	424.87	0.018617	484.89	0.018698	424.87	0.00%	0.00%	0.00%	0.00%
4	0.005948	408.39	0.005870	335.39	0.005948	408.39	0.005870	335.39	0.00%	0.00%	0.00%	0.00%
	0.020386	469.45	0.019706	412.96	0.020386	469.45	0.019706	412.96	0.00%	0.00%	0.00%	0.00%
5	0.005378	227.81	0.005545	254.88	0.005691	324.01	0.005640	315.71	5.82%	42.23%	1.71%	23.87%
	0.041196	244.14	0.022223	321.66	0.022124	390.26	0.022223	386.36	-46.29%	59.85%	0.00%	20.12%
6	0.005143	202.85	0.005106	198.93	0.005283	264.93	0.005250	261.03	2.73%	30.60%	2.81%	31.22%
	0.049102	216.37	0.050360	211.96	0.047834	280.89	0.048820	276.47	-2.58%	29.82%	-3.06%	30.43%
7	0.004914	179.06	0.004886	176.17	0.005077	241.30	0.005052	238.44	3.32%	34.76%	3.40%	35.35%
	0.057126	189.69	0.058161	186.47	0.053992	254.20	0.054767	250.99	-5.49%	34.01%	-5.84%	34.60%
R	0.004664	154.08	0.004643	152.04	0.004857	216.65	0.004839	214.65	4.13%	40.61%	4.21%	41.18%
	0.066450	162.03	0.067246	159.8	0.060840	226.67	0.061413	224.45	-8.44%	39.89%	-8.67%	40.46%

All joints follow the sequence "beam flexural hinging – joint shear failure – column flexural hinging", and the details are not repeated in this section.

Table 7- 49: Sequence of mechanisms determined by Portal Frame method

Original component strength		Changed component strength	
Level	**Mechanism**	**Level**	**Mechanism**
Level 5	Beam yield (structure "First Yield" state)	Level 2	Beam yield (structure "First Yield" state)
Level 2	Interior beam flexural hinging	Level 3	Interior beam flexural hinging
Level 5	Exterior beam flexural hinging	Level 6	Interior beam flexural hinging
Level 5	Interior beam flexural hinging	Level 4	Interior beam flexural hinging
Level 3	Interior beam flexural hinging	Level 5	Beam yield
Level 4	Interior beam flexural hinging	Level 5	Exterior beam flexural hinging
Level 6	Beam yield	Level 1	Interior beam flexural hinging
Level 6	Exterior beam flexural hinging	Level 5	Interior beam flexural hinging
Level 6	Interior beam flexural hinging	Level 6	Beam yield
Level 2	Beam yield	Level 7	Interior beam flexural hinging
Level 2	Exterior beam flexural hinging	Level 6	Exterior beam flexural hinging
Level 2	Joint shear failure	Level 2	Beam yield
Level 3	Beam yield	Level 2	Exterior beam flexural hinging
Level 3	Joint shear failure	Level 2	Joint shear failure
Level 1	Joint shear failure	Level 3	Beam yield
Level 1	Beam yield	Level 3	Joint shear failure
Level 3	Exterior beam flexural hinging	Level 1	Joint shear failure
Level 4	Joint shear failure	Level 1	Beam yield
Level 1	Exterior beam flexural hinging	Level 3	Exterior beam flexural hinging
Level 4	Beam yield	Level 4	Joint shear failure

Level 1	Interior beam flexural hinging	Level 1	Exterior beam flexural hinging
Level 4	Exterior beam flexural hinging	Level 4	Beam yield
Level 6	Interior column flexural hinging	Level 4	Exterior beam flexural hinging
Level 5	Joint shear failure	Level 5	Joint shear failure
Level 7	Beam yield	Level 7	Beam yield
Level 7	Exterior beam flexural hinging	Level 7	Exterior beam flexural hinging
Level 7	Interior beam flexural hinging	Level 6	Joint shear failure
Level 6	Joint shear failure	Level 1	Interior column flexural hinging
Level 5	Interior column flexural hinging	Level 2	Interior column flexural hinging
Level 1	Interior column flexural hinging	Level 3	Interior column flexural hinging
Level 7	Interior column flexural hinging	Level 6	Interior column flexural hinging
Level 2	Interior column flexural hinging	Level 4	Interior column flexural hinging
Level 3	Interior column flexural hinging	Level R	Interior beam flexural hinging
Level 4	Interior column flexural hinging	Level 7	Joint shear failure
Level 7	Joint shear failure	Level 5	Interior column flexural hinging
Level 5	Exterior column flexural hinging	Level 7	Interior column flexural hinging
Level 5	Column shear failure	**Level 1**	**Column shear failure**
Level 1	Column shear failure	Level R	Beam yield
Level R	Beam yield	Level 2	Column shear failure
Level 2	Column shear failure	Level R	Exterior beam flexural hinging
Level R	Interior column flexural hinging	Level 5	Column shear failure
Level R	Exterior beam flexural hinging	Level 3	Column shear failure
Level 6	Exterior column flexural hinging	Level 4	Column shear failure
Level R	Interior beam flexural hinging	Level 6	Column shear failure
Level 3	Column shear failure	Level R	Joint shear failure
Level 6	Column shear failure	Level 6	Exterior column flexural hinging
Level 4	Column shear failure	Level R	Interior column flexural hinging
Level 7	Exterior column flexural hinging	Level 2	Exterior column flexural hinging
Level R	Joint shear joint	Level 1	Exterior column flexural hinging
Level 2	Exterior column flexural hinging	Level 3	Exterior column flexural hinging
Level 1	Exterior column flexural hinging	Level 5	Exterior column flexural hinging
Level 3	Exterior column flexural hinging	Level 4	Exterior column flexural hinging
Level 7	Column shear failure	Level 7	Column shear failure
Level 4	Exterior column flexural hinging	Level 7	Exterior column flexural hinging
Level R	Exterior column flexural hinging	Level R	Exterior column flexural hinging
Level R	Column shear failure	Level R	Column shear failure

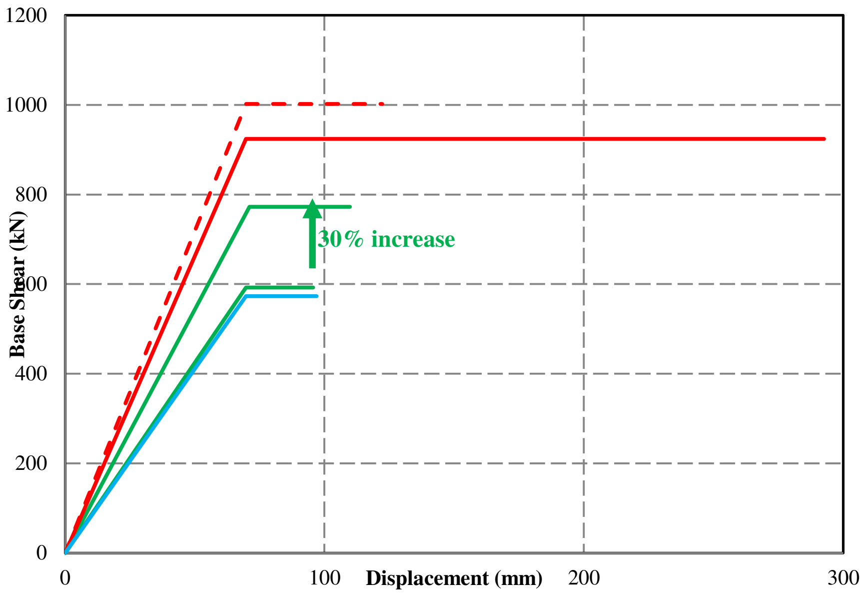


Figure 7- 24: Comparison of the pushover curves computed by the improved SLaMa with Portal Frame Method before and after component strengths changed

Table 7- 50: Summary of base shears and ultimate displacements calculated by the improved SLaMa with Portal Frame Method using the original and the changed component strengths

Capacity	Original interior beam strengths	Reduced interior beam strengths	Ratio of Increase
Base shear (kN)	593	772	30.30%
Ultimate displacement (mm)	96	110	14.81%

In Table 7- 49, the comparison of the sequence of mechanisms determined by Portal Frame Method with the application of the original and the changed component strengths are shown, and the bi-linear pushover curves computed are shown in Figure 7- 24. The differences found are summarised as following:

- In general, quite a lot of changes were found when applying the changed component strengths.
- Joint shear failure mechanisms were found to be delayed when applying the changed component strengths.
- Column flexural hinging mechanisms were also found to be delayed when applying the changed component strengths.
- The most critical mechanism was assessed to the “column shear failure at level 1”, which was found to be very different from the original critical mechanism – “exterior column flexural hinging at level 5”.
- As shown in Figure 7- 24 and Table 7- 50, the base shear capacity calculated using the changed component strengths was found to be 30% higher than the original base shear capacity. The ultimate displacement capacity calculated using the changed component strengths was found to be 14.81% higher than the original ultimate displacement.

7.5. Alternative Building Case Studies (Potential Parametric Study)

Modelling sheets of additional building case studies are attached as Appendix A14. The modelling sheets collect information such as basic building data, elevation and plan drawings of the critical structural systems, summaries of section profiles for the critical structural systems, and calculation of loading cases. All the building case studies selected in this research are frame-type buildings from the Refined CHCH building database knowledge level 2 (see Section 6.3). Due to the restricted timeframe of this research, detailed assessment procedures were completed only for a few cases. In following paragraphs, the assessment results of one alternative case study – Building No.46 – are presented and briefly discussed. The modelling sheets, together with assessment calculation spreadsheets, may provide a good source of information for future researchers. It is worth noting that the information provided in the modelling sheets and calculation spreadsheets requires further examination and modification.

In Table 7- 51, a summary of flexural capacities of beams and columns in Frame D of Building No.46 is presented. By performing shear checks for the components following the same procedures in assessing Building No.21, it was found that the frame is unlikely to prone to shear failure. In Figure 7- 25, the results from the evaluation of strength hierarchy at subassembly level are presented, and it can be deduced that the sequence of global mechanisms should following the order "beam flexural hinging – joint shear failure – column flexural hinging". This prediction was confirmed by the results from Portal Frame Method, as shown in Table 7- 52. In Figure 7- 26, the upper bound and the lower bound of lateral load capacity of the frame are shown, computed by following the same procedures applied for Building No.21. The lower bound was determined by assuming a column sidesway mechanism occurring at level 1, and the upper bound was determined by assuming a beam sidesway mechanism. By following Portal Frame Method and properly applying the determined displaced shape in displacement-based assessment procedure, a bi-linear pushover curve of the frame with mixed sidesway mechanism (which is more likely to be the real response) was computed, bounding between the upper and the lower bounds.

Table 7- 51: Summary of beams and columns yielding and ultimate flexural moment capacity (based on nominal material strengths) (without consideration of axial load on columns) of Frame D in Building No.46

Level	Beam				Column			
	Exterior		Interior		Exterior		Interior	
	My	My	My	My	My	My	My	My
	Mu	Mu	Mu	Mu	Mu	Mu	Mu	Mu
1	0.003959	160.96	0.003959	160.96	0.003620	273.65	0.003546	301.00
	0.068718	191.74	0.068718	191.74	0.024814	419.08	0.040514	386.26
2	0.003959	160.96	0.003959	160.96	0.003620	273.65	0.003546	301.00
	0.068718	191.74	0.068718	191.74	0.024814	419.08	0.040514	386.26
3	0.003751	111.22	0.003751	111.22	0.003620	273.65	0.003546	301.00
	0.087886	118.27	0.087886	118.27	0.024814	419.08	0.040514	386.26
R	0.003751	111.22	0.003751	111.22	0.003620	273.65	0.003546	301.00
	0.087886	118.27	0.087886	118.27	0.024814	419.08	0.040514	386.26

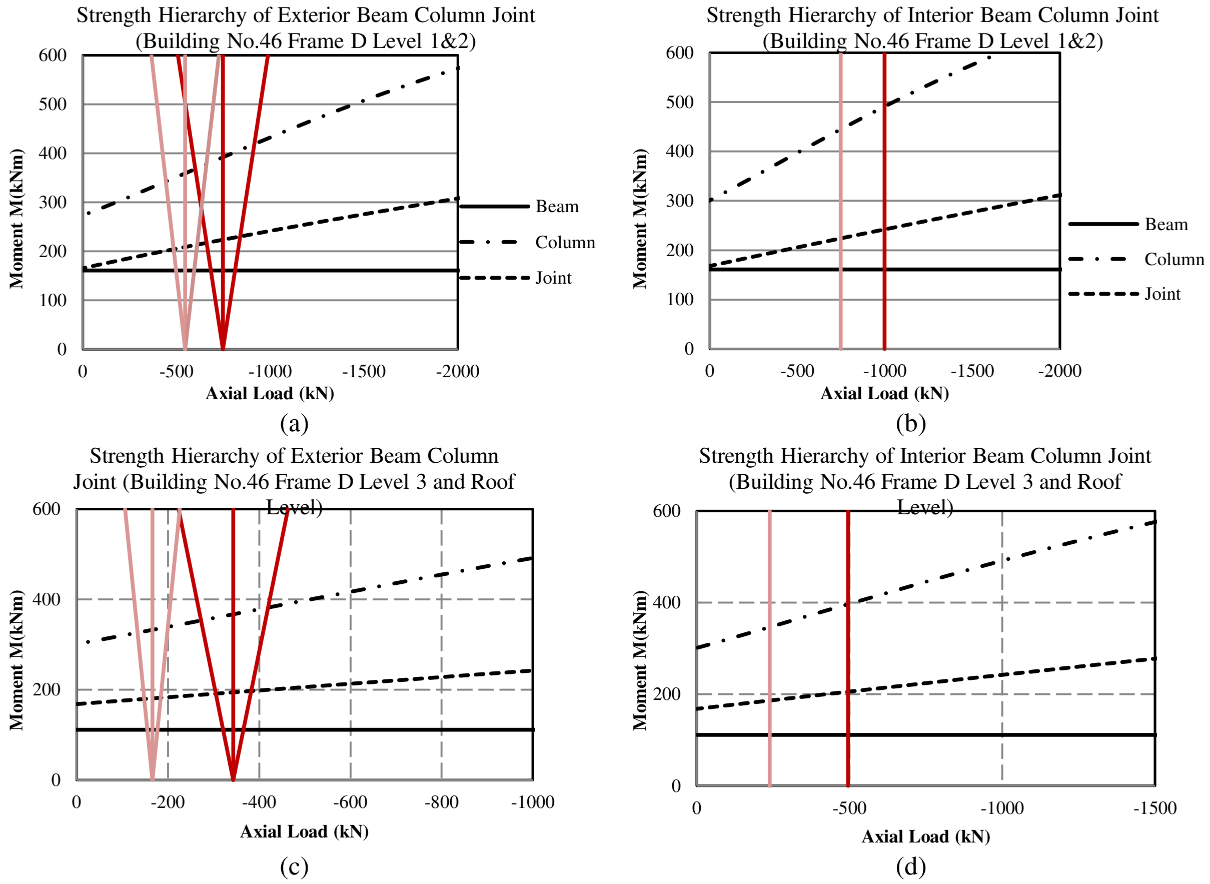


Figure 7- 25: Strength hierarchy evaluation for beam column joints in Frame D of Building No.46: (a) exterior joints at level 1 and 2; (b) interior joints at level 1 and 2; (c) exterior joints at level 3 and roof; (d) interior joints at level 3 and roof

Table 7- 52: Sequence of mechanisms determined by Portal Frame method

Level	Mechanism	Storey Shear (kN)	Base Shear (kN)	Local displacement (mm)	Global displacement (mm)
Level 3	Beam yield	245	350	1.335	5.341
Level 1	Beam yield	354	354	1.932	5.938
Level 3	Exterior beam flexural hinging	260	372	1.420	6.022
Level 2	Beam yield	354	394	1.932	6.620
Level 3	Interior beam flexural hinging	277	396	2.529	7.729
Level 1	Exterior beam flexural hinging	422	422	2.302	8.099
Level 1	Interior beam flexural hinging	450	450	4.100	9.897
Level 2	Exterior beam flexural hinging	422	469	2.302	10.267
Level 2	Interior beam flexural hinging	450	500	4.100	12.065
Roof	Beam yield	245	612	1.335	12.150
Roof	Exterior beam flexural hinging	260	651	1.420	
Roof	Interior beam flexural hinging	277	693	2.529	13.259
Level 1	Joint shear failure	752	752		25.162
Level 2	Joint shear failure	697	774		29.739
Level 1	Interior column flexural hinging	906	906	47.228	56.387
Level 3	Joint shear failure	641	915		60.555
Level 2	Interior column flexural hinging	906	1006	47.228	99.515
Level 3	Interior column flexural hinging	906	1294	47.228	144.214
Roof	Joint shear failure	577	1441		
Level 1	Column shear failure	1644	1644		
Level 2	Column shear failure	1644	1827		
Level 1	Exterior column flexural hinging	1844	1844	41.553	
Level 2	Exterior column flexural hinging	1844	2049	41.553	
Roof	Interior column flexural hinging	906	2264	47.228	
Level 3	Column shear failure	1644	2349		
Level 3	Exterior column flexural hinging	1844	2635	41.553	
Roof	Column shear failure	1644	4111		
Roof	Exterior column flexural hinging	1844	4611	41.553	

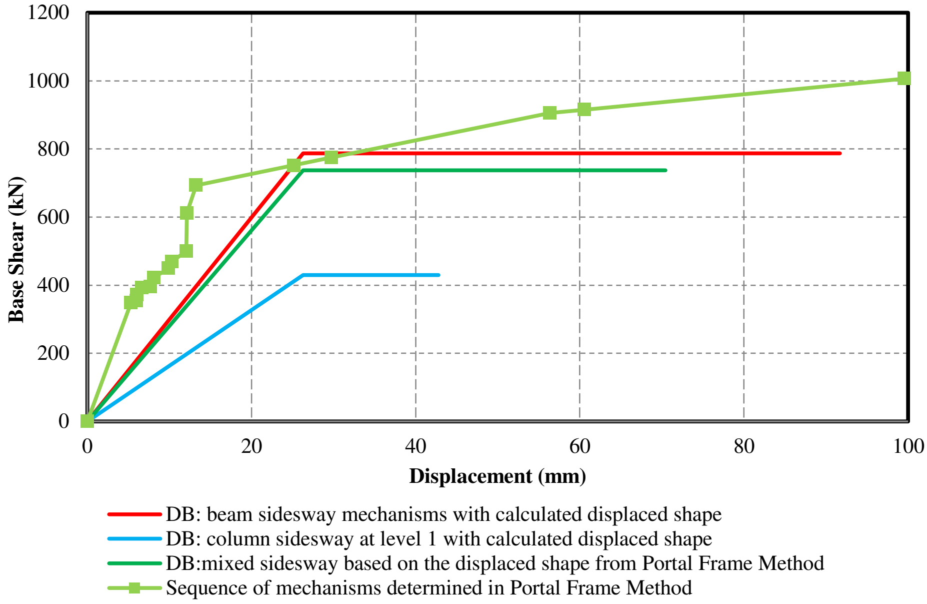


Figure 7- 26: Pushover curves computed for Frame D of Building No.46

As discussed in Section 5.5.3 and Section 6.3.4, in order to generate component analysis models and global structure models, parametric studies associated with different types of components and also the global structures should be carried out. As mentioned in previous, due to the restricted timeframe, parametric studies on the components and global structures have not completed. Table 7- 53, shown in the end of this section, is provided to explain the future work regarding parametric study.

For the selected building representatives, both simplified and comprehensive assessment procedures should be carried out, and pushover curves (1) lower bound (from the improved SLaMa); (2) upper bound (from the improved SLaMa); (3) from simplified assessment procedures (i.e. the improved SLaMa); (4) from comprehensive assessment procedures (i.e. involving numerical modelling) should be computed. If multiple buildings are assessed for one building typology, a generalised pushover curve may be determined. And as mentioned in Section 5.5.3, the response of a reinforced concrete building can be directly predicted by referring to the generalised pushover curve of the building representatives that have the most alike properties or characteristics to the building case under assessment, without involving too much effort in data collection, complicated calculation and numerical modelling.

During assessing the selected building representatives, generalised component models (as mentioned in Section 5.5.3, i.e. force-deformation curve, component "pushover" curve) may be computed. Hence, the response of an element (e.g. beam, column, joint, wall or others) can be directly predicted by referring to the generalised curve of the components that have the most alike properties or characteristics. The predicted responses of elements can then be applied as inputs to assess the global response of the structure. In order to accomplish the goal of establishing generalised component models and global structure models, researchers from specific disciplines, such as beam, column, joint, wall, masonry, residual capacity, etc. need to be involved.

As shown in Table 7- 53, reinforced concrete buildings from the Refined CHCH Building Database knowledge level 2 are presented. For building No.39, No.45, No.21, No.46, No.75, No.7, No.38, No.56, No.74 and No.31, modeling sheets are attached as appendices (Appendix A14), and assessment calculation spreadsheets are provided but will require future modifications. For building No.5, No.10, and No.41, despite the current structural drawings (which have missing parts and pages), more sufficient building data are required.

Table 7- 53: Potential parametric study matrix

Building Typology	Parameter SET1	Parameter SET2	Parameter SET3	Parameter SET4	Parameter SET5	Parameter SET 6
Frame (pre/incl 70s) low rise (1-3 storey)						
Frame (pre/incl 70s) mid-rise (4-7 storey)	Building No.39 (Trade Union Centre)	Building No.45 (Strategy House)	Building No.44 (Old CHC City Council Building)	Building No.99 (George Hotel)		
Frame (pre/incl 70s) high rise (8+ storey)	Building No. 21 (Securities House)					
Frame (post70s) low rise (1-3 storey)						
Frame (post 70s) mid rise (4-7 storey)	Build No. 46 (Alliance Insurance)	Building No.75 Riverlands House)	Building No.5 (Amuri Courts)			
Frame (post 70s) high rise (8+ storey)	Building No.7 (Brannigans)	Building No.38 Victoria Tower)	Building No.56 (Drexel Restaurant and Above)	Building No.74 (BBQ House)	Building No.13 (Clarendon Tower)	Building No.10 (Crown Plaza)
Shear Wall (pre/incl 70s) low rise (1-3 storey)	Building No. 41 (CHC City Town Hall)					
Shear Wall (pre/incl 70s) mid rise (4-7 storey)						
Shear Wall (pre/incl 70s) high rise (8+ storey)	Building No. 31 (AMI Building)					
Shear Wall (post70s) low rise (1-3 storey)						
Shear Wall (post 70s) mid rise (4-7 storey)	Building No. 71 (Wilson Parking)					
Shear Wall (post 70s) high rise (8+ storey)	Building No. 36 (Terrace on the Park)					

More building cases should be included in order that for each of building types, there is at least one building representative. For these building representatives:

(1) Simplified assessment procedures should be carried out, and pushover curves with upper and lower bounds should be determined.

(2) Numerical modeling should be carried out, in order to verify the results from simplified procedures.

The classification of buildings can be more sophisticated (see Section 6.3) if more building examples are considered.

Building cases under the same building typology category may have similar member section properties

	Parameter Set No. #		
Building Typology Frame or wall, Year of construction, Low/Mid/High Rise, etc.	**Concrete strength** **Reinforcing steel strength** **Concrete ultimate compressive strain** **Beam Geometry** **Column Geometry** **Beam vertical reinforcement ratio** **Beam transverse reinforcement ratio** **Column vertical reinforcement ratio** **Column transverse reinforcement ratio** **Etc.**	If the variations of these properties do NOT influence the failure mechanism nor the sequence of mechanisms, they only affect the bandwidth within the boundaries.	However, if variations of these properties do influence the failure mechanism or significantly influence the sequence of mechanisms, then things can get complicated.

CHAPTER 8 Numerical Modelling

8.1. Purpose of Numerical Modelling

The purpose of numerical modelling is stated as following:

- To validate the deficiencies of the simplified analytical (i.e. "by-hand") approach (i.e. SLaMa) that is currently adopted in NZSEE 2006. These deficiencies are discussed in the previous chapters.
- To validate the improvements proposed to the simplified analytical approach, i.e. by including procedures of (1) evaluating strength hierarchy; (2) determining the lower and upper bounds of the lateral load capacity of the structure; (3) determining the sequence of mechanisms by Portal Frame Method (incl. Evaluation of Strength Hierarchy); or (4) adopting the component analysis model and global structure model

In aiming to achieve the purposes stated above, the case study structure, Frame 1 of Building No.21 was modelled, and the modelling results (e.g. a pushover curve, prediction of the mechanisms sequence, prediction of the potential damages) was compared to the results obtained from the current and the improved simplified analytical approaches.

In Section 8.2 and Section 8.3, the details associated with establishing the component models and developing the model for the entire structure are presented. Section 8.4 provides a brief explanation of the nonlinear analyses adopted, and the results generated from the analyses are shown in Section 8.5. Following the presented results from numerical modelling, the comparison of the numerical modelling results to the results obtained from the various approaches is also shown, with a discussion of the validation of the deficiencies of the current simplified approach as well as the proposed improvements to this simplified approach.

8.2. Literature Review of the Plasticity Model for Beam-Column Joint

One of the most critical modelling issues is whether the beam-column joint can be properly modelled. The proper joint model should have the shear resistance and shear deformation developed in joint region well defined; otherwise, the response of the structure in a seismic event cannot be appropriately predicted. This section gives a summary of the beam-column joint models from literature review.

As shown in Figure 8- 1, a simple equivalent moment rotational spring was proposed by Pampanin *et al.* (2002), and the linear or nonlinear behaviour of the joint can be modelled by applying this model. The characteristics of such spring model can be summarised as:

- The spring converges into one node – the centre of the joint region. During modelling, it is assumed that the spring element should be set as "zero-length", i.e. very small length, for instance 0.001m (1mm).
- The moment-curvature relationship of the rotational spring should be derived based on the experimental principal tensile stress vs. joint shear deformation relationship. A summary of the principal tensile stress vs. joint shear deformation relationships obtained from experimental research in the past twenty years is provided in Section 5.3.3, and the relationship adopted in the proposed model is referred to Figure 5- 4 or Figure 5- 5 (Section 5.3.3). It is worth noting that the k values specified in Figure 5- 4 and Figure 5- 5 (also, Figure 8- 2) are only applicable for joints with specific types of reinforcement and anchorage. Therefore, further tests and research work is still required to define or refine k values for the joints with various reinforcement and anchorage types.
- The equivalent joint spring moment corresponding to a defined level of principal tensile (or compressive) stress in the joint (e.g. first cracking, or high damage level) should be calculated following the procedure presented in Section 5.3.3. As discussed in Section 5.3.3, only the concrete mechanism contributed to joint shear resistance is considered.
- The joint shear distortion mechanism is assumed to become dominant at higher level of deformation, i.e., it is predominant when the joint is subjected to flexural behaviour. The joint shear deformation is assumed to be equal to the spring rotation. (Pampanin *et al.,* 2002)
- The cyclic behaviour should be modelled by applying an appropriate hysteretic rule accounting for the "pinching" effect due to slip of the reinforcement and shear cracking in the joint, for instance, Pampanin Reinforced Concrete Beam-Column Joint Hysteresis (IHYST=44 in RUAUMOKO), or Wayne Stewart Degrading Stiffness Hysteresis (IHYST=9 in RUAUMOKO).

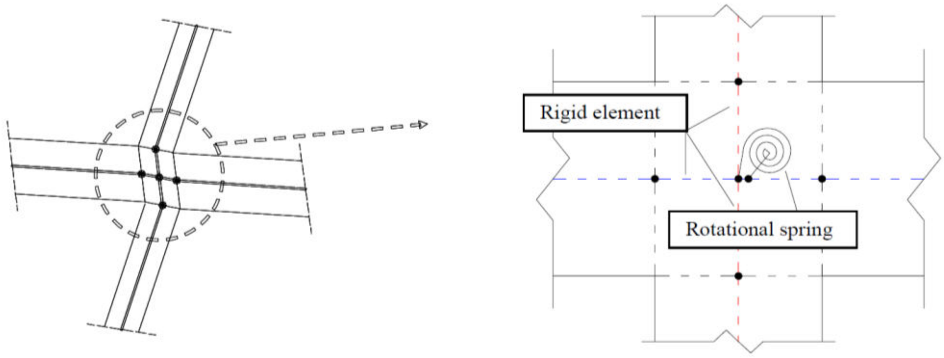


Figure 8- 1: Simple lumped plasticity model for beam-column joints with a close up view of the panel zone region (Pampanin et al, 2002)

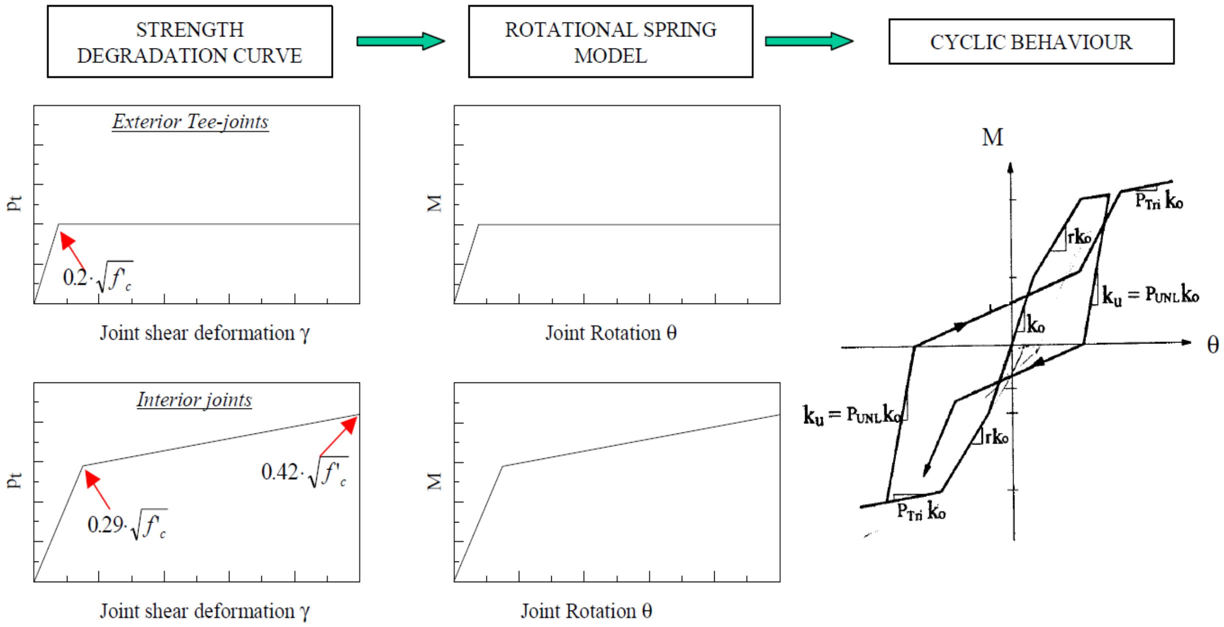


Figure 8- 2: Monotonic and cyclic behaviour of the shear hinge model (Pampanin et al, 2002)

Galli (2006) mentioned that though this simple lumped plasticity joint model is easy and convenient to apply, however, it fails to take account for the effect of axial load on joint shear resistance. Hence, he proposed a modification to the model, which is shown in Figure 8- 3.

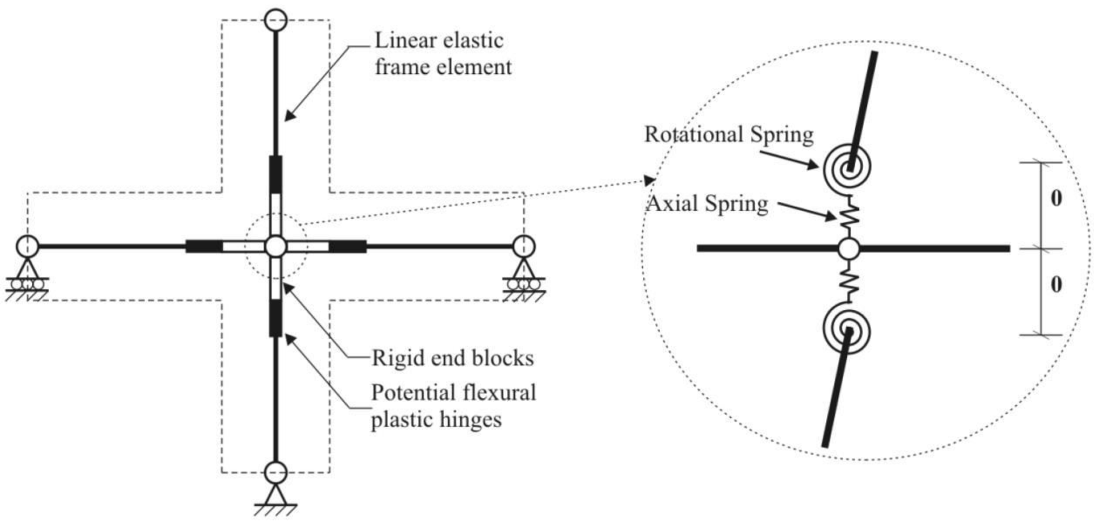


Figure 8- 3: Modified joint model representation (Trowland, 2003) (Galli, M., Evaluation of the Seismic Response of Existing R.C. Frame Buildings with Masonry Infills, 2006)

In this modified model, as illustrated in Figure 8- 3, the joint resistance and shear deformation is modelled by two rotational springs, interposed between the beam connection node and the upper column and the lower column, respectively. Each of these rotational springs takes half of the joint strength and stiffness. The axial load transmitted through the joint is modelled by applying an axial spring, interposed between the split rotational springs. Though the modification to take account for the effect of axial load was proposed, the model still fails to appropriately model the steel reinforcement contribution.

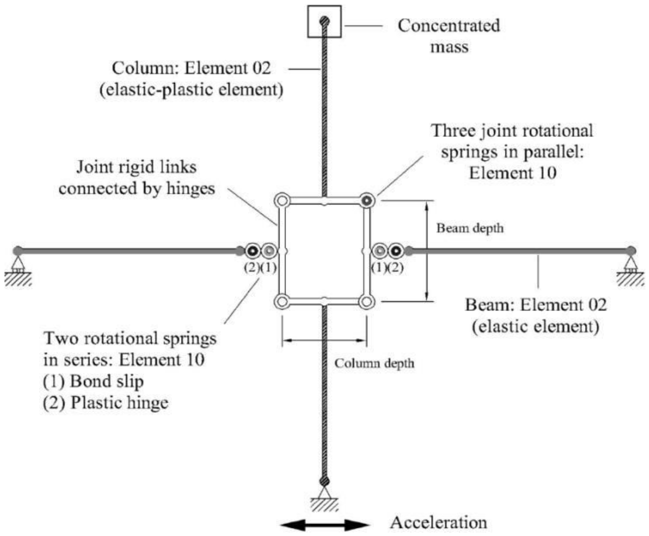


Figure 8- 4: Alternative beam-column joint model adopted by Shin, et al., 2004

Alternative models, for instance, the model developed by Shin, *et al.*, shown in Figure 8- 4, enable properly modelling of steel reinforcement contribution to joint shear resistance. In Figure 8- 5 and Figure 8- 6, the hysteretic rule and the joint principal stress-shear strain relationship, applied in Shin's model, are presented. In Figure 8- 7, a number of alternative sophisticated models of beam-column joint are also presented. However, the applying these models may requires higher analysis program capability and more research efforts.

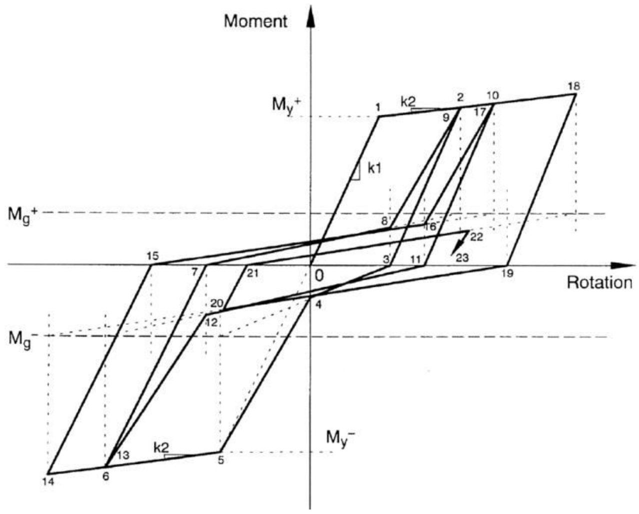


Figure 8- 5: Alternative hysteretic behaviour of the analysed joint element adopted by Shin, et al, 2004

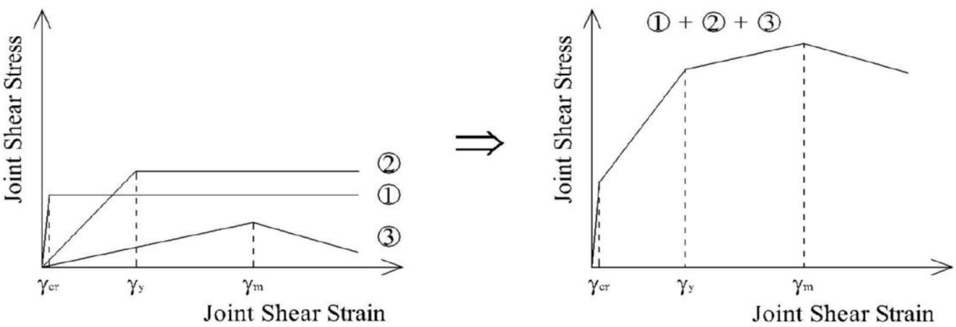


Fig. 14 Combination of three bilinear joint springs in parallel

Figure 8- 6: Alternative joint principal stress-shear strain adopter by Shin, et al, 2004

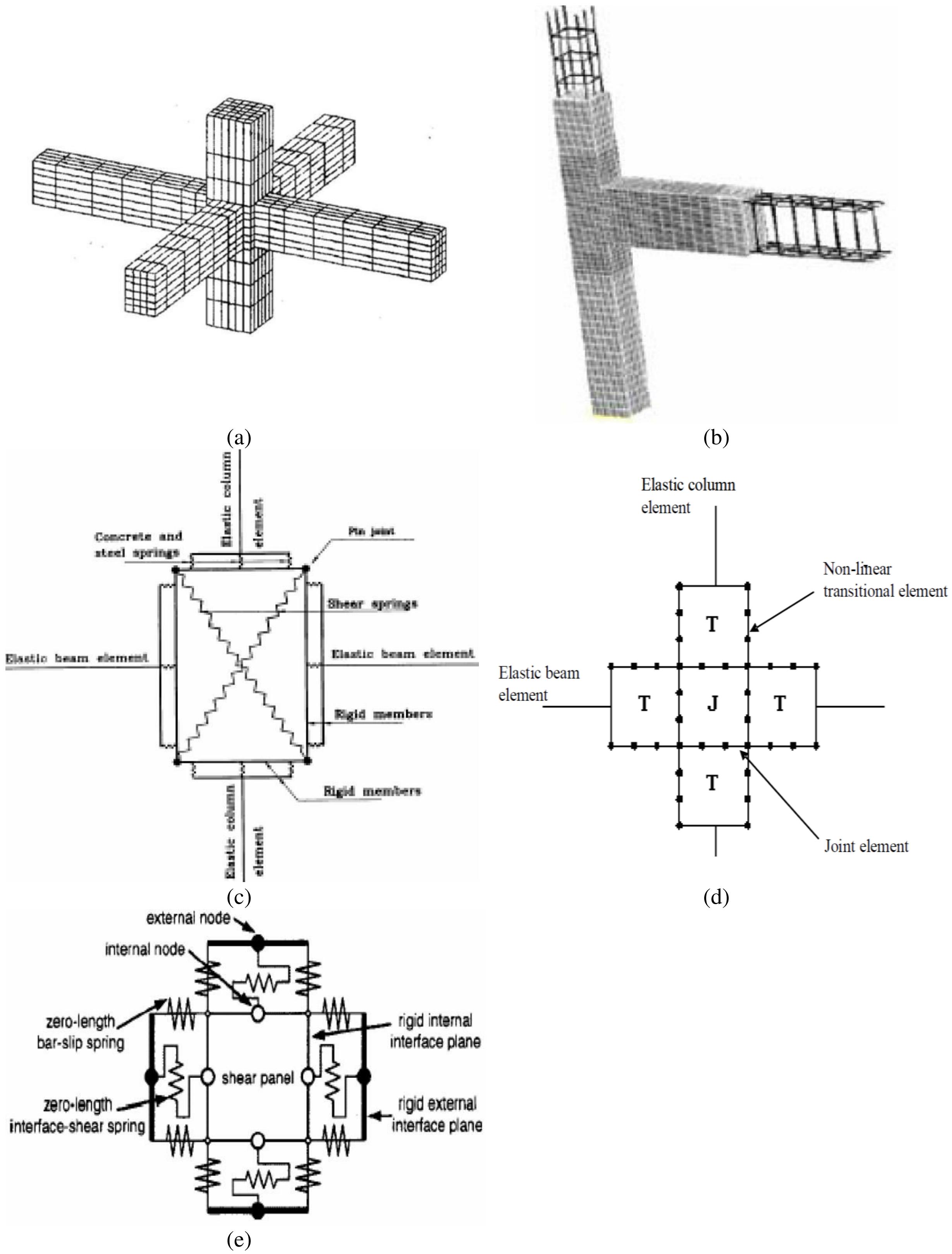


Figure 8- 7: Alternative sophisticated beam-column joint models (a) Finite element model adopted by Nagai, 1996; (b) Model of test specimen adopted by Eligehausen et al, 2006;(c) Multi-spring model proposed by Youssef and Ghobarah, 2001;(d) Joint model proposed by Elmorsi, Kianoush and Tso, 2000; (e) Reinforcement concrete beam-column joint model adopted by Lowes et al., 2003

8.3. Establishment of Model and Determination of Inputs

It was determined that RUAUMOKO 2D was used to carry out nonlinear pushover analysis. From Section 8.3.1 to 8.3.3, brief explanations of component models are given, with the determination of the modelling inputs shown.

8.3.1. Joint Model

The beam-column joint model proposed by Galli was used in numerical modelling. As discussed in Section 8.2, Galli's model is superior to the simple plasticity model by taking the effect of the axial load into consideration while still keeps simplicity. However, it has been emphasised in Section 8.2 that the reinforcing steel contribution to joint shear resistance cannot be properly analysed by the Galli's Model. In Frame 1 of Building No.21, it was found that no additional transverse reinforcements were designed in the joint regions, as illustrated in Figure 8- 9. Therefore, as stated in Section 7.3.3 and Section 7.3.4, the reinforcing steel contribution to the shear resistance in the joint regions are negligible compared to concrete strut contribution. The k value was assumed as 0.42 during calculating the shear principal stress, as discussed in Section 7.3.4.

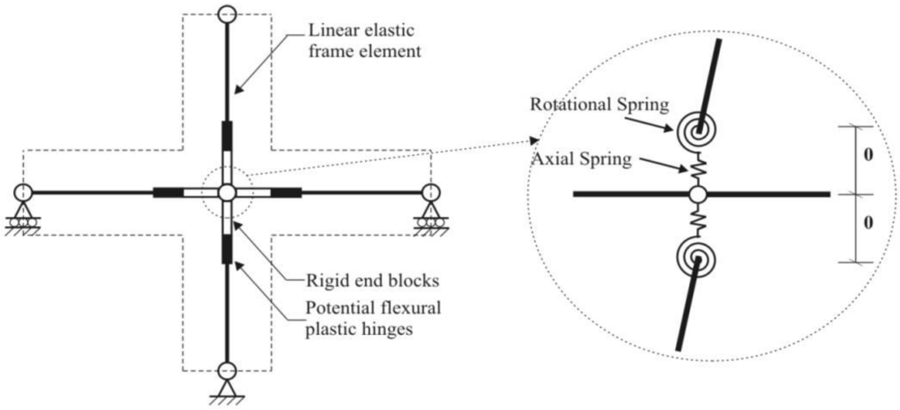


Figure 8- 8: Adoption of joint model from literature

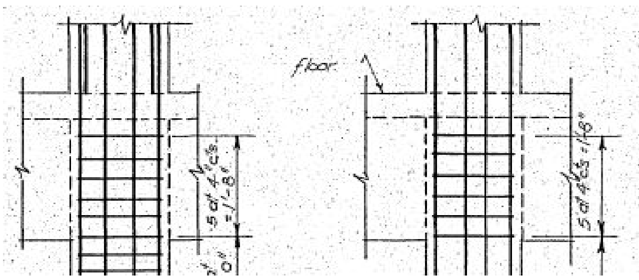


Figure 8- 9: Transverse reinforcing details found in joint region for Frame 1 of Building No.21

The basic section properties of the joint springs should be determined by applying the following:

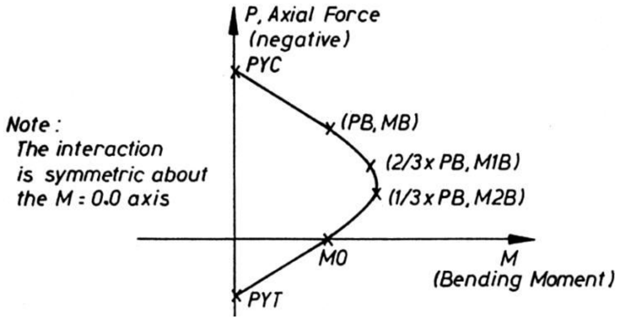
$$K_j = G_c\left(\frac{jd_b H}{H_c - jd_b}\right)A_e$$

$$K_A = \frac{E_c A_e}{L}$$

K_j = Joint rotational stiffness
K_A = Joint axial stiffness
G_c = Concrete shear modulus, and is assumed to be $G_c = \frac{E_c}{2(1+v)}$,and this is also assumed for beam and column elements
E_c = Concrete elastic modulus, and is 23209160kPa (23.2MPa) found for Frame 1 of Building No.21
j = 0.9 as stated in Section 5.3.3
v = Concrete Poisson's Ratio, and is assumed to be 0.2
H_c = Interstorey height/column height
d_b = Beam depth
A_e = Joint effective area
L = Half of the joint panel height

The joint rotational and axial stiffness was estimated as 1.56E+06kNm/rad and 1.32E+07kN/m respectively. The stiffness should be split evenly to the two rotational springs.

Apart from the top level joint springs, the spring characteristics were defined by the equivalent moment-axial load relationships. The curves were computed based on the six points, (PYT, 0), (0, M0), (⅓PB, M2B), (⅔PB, M1B), (PB, MB) and (PYC, 0), as shown in Figure 8- 10. The determination of these six points followed the procedure presented in Section 5.3.3. For the top level joints, no axial load influence was considered; hence ITYPE=1 was applied for the top level joint springs, assuming no interactions between X-, Y-, and Z- components. Table 8- 1 gathers the calculated values of the parameters required to define the characteristics for exterior and interior joints from level 1 to roof level.



PYT = Axial tension yield force (>0)
PYC = Axial compression yield force (<0)
$FY+$ = Positive yield force in shear (>0)
$FY-$ = Negative yield force in shear (<0)
PB = Axial compression force at B (<0)
MB = Yield moment at B
$M1B$ = Yield moment at P=(2/3)PB
$M2B$ = Yield moment at P=(1/3)PB
$M0$ = Yield moment at P=0

Figure 8- 10: Joint equivalent moment-axial load relationship (i.e. yield surface relationship for ITYPE=4, axial force-yield moment interaction) (RUAUMOKO2D Manual, Volume 2.12f)

Table 8- 1: Summary of the six points to compute joint equivalent moment-axial load curve

Joint	PYT (kN)	PYC (kN)	PB (kN)	MB (kNm)	⅔PB (kN)	M1B (kNm)	⅓PB (kN)	M2B (kNm)	M0 (kNm)
Level 1 Exterior	431	-3072	-1320	278	-880	240	-440	196	138
Level 1 Interior	431	-2576	-1072	258	-715	225	-357	187	138
Level 2 Exterior	431	-2651	-1110	261	-740	227	-370	188	138
Level 2 Interior	431	-2309	-939	246	-626	216	-313	182	138
Level 3 Exterior	431	-2285	-927	245	-618	215	-309	181	138
Level 3 Interior	431	-2042	-805	234	-537	207	-268	176	138
Level 4 Exterior	431	-1918	-743	227	-496	202	-248	173	138
Level 4 Interior	431	-1774	-671	221	-448	197	-224	170	138
Level 5 Exterior	431	-1592	-580	216	-387	194	-193	170	141
Level 5 Interior	431	-1505	-537	212	-358	191	-179	168	142
Level 6 Exterior	431	-1282	-426	194	-284	177	-142	159	138
Level 6 Interior	431	-1234	-402	192	-268	176	-134	158	138
Level 7 Exterior	431	-995	-282	177	-188	165	-94	152	138
Level 7 Interior	431	-961	-265	176	-176	164	-88	152	138

Joint	FX+ (kN)	FX- (kN)	FY+ (kN)	FY- (kN)	MZ+ (kNm)	MZ- (kNm)
Level R Exterior	-1.32E+07	1.32E+07	1.32E+07	-1.32E+07	158	-158
Level R Interior	-1.32E+07	1.32E+07	1.32E+07	-1.32E+07	157	-157

It is worth recognizing that the ductility-based strength degradation was assigned to the exterior joint model according to Galli's research, based on the results from experiment test on joint subassemblies.

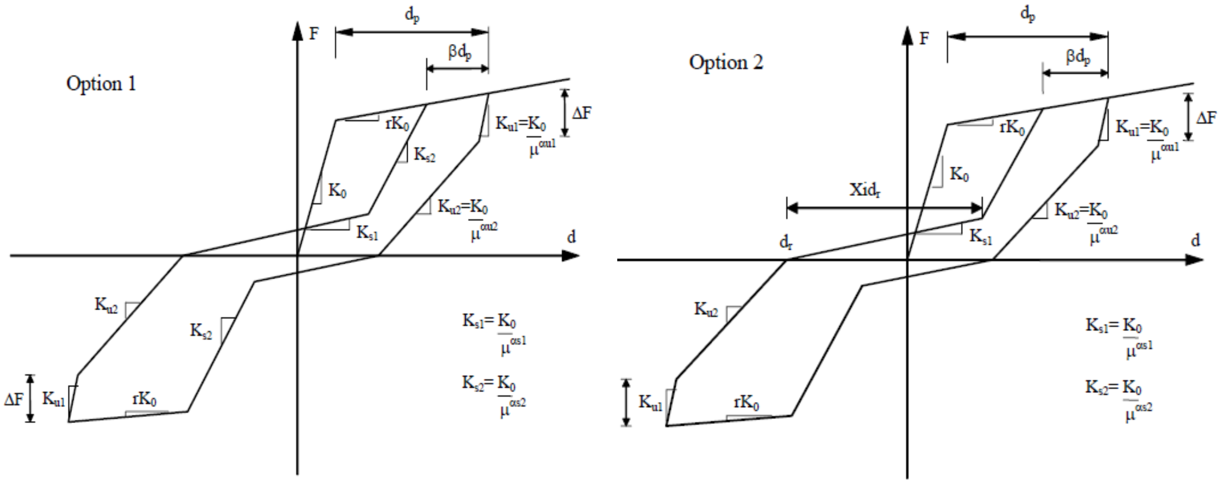
Table 8- 2: Summary of strength degradation parameters

Parameters	DUCT1	DUCT 2	RDUCT
Values	1	161	0.8

Where:

DUCT1	=	Ductility at which degradation begins, or the cycle number that the strength starts to reduce
DUCT2	=	Ductility at which degradation stops, or the cycle number where the strength reaches the residual value
RDUCT	=	Residual strength as a fraction of the initial yield strength

Pampanin Hysteresis (i.e. IHYST=44) Option 2, as shown in Figure 8- 11, was applied to model the joint behaviour under cyclic loading. The values of the hysteretic parameters should be determined and calibrated according to the results of experimental testing on beam-column joint subassemblies. The calibrated values proposed by Galli (Table 8- 3) were used during modelling. However, it should be recognised that the subassemblies in the testing had different characteristics with the joints in the case study frame. Hence, the application of these calibrated values introduced uncertainty to the modelling results.



(a) (b)

Figure 8- 11: Pampanin hysteretic rule (IHYST=44) (a) Option 1-reloadin power factor; (b) Option 2-reloading slip factor

Table 8- 3: (a) Parameters needed to defined hysteresis rule adopted for joint members; (b) Calibration of the hysteretic rule parameters for the beam-column subassemblies

Option 1	Option 2	Related Process
α_{s1}	α_{s1}	reload
α_{s2}	X_i	reload
α_{u1}	α_{u1}	unload
α_{u2}	α_{u2}	unload
ΔF	ΔF	unload
β	β	reload

(a)

Parameter	Specimen			
	T1	T2	L1	C2
α_{s1}	1.2	1.3	1.3	1.2
X_i	1.5	1.3	1.4	1.3
α_{u1}	-0.1	-0.1	-0.1	-0.1
α_{u2}	0.9	0.8	0.8	0.95
ΔF	30	30	20	30
β	-0.2	-0.3	-0.1	0

(b)

8.3.2. Beam and Column Model

The beams and columns were modelled as one-dimensional elements with lumped plasticity (illustrated as in Figure 8- 12) in the end regions with associated component characteristics, for instance, moment-curvature relationships for beams, moment-axial load interaction relationships for columns, etc.

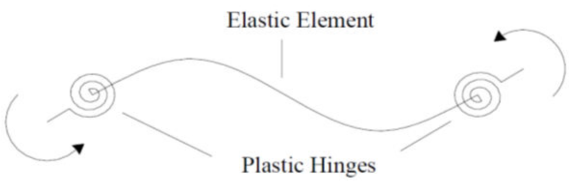


Figure 8- 12: Lumped plasticity element

For beam elements, Table 8- 4 gives a summary of the determined beam characteristics. It is worth noting that for each of the beam and column elements, same yielding surface was assumed for two ends (i.e. end 1 and end 2).

Table 8- 4: Summary of beam characteristics

Beams	A (m)	AS (m2)	I (m4)	RF	H1 (m)	H2 (m)	MY1+ (kNm)	MY1- (kNm)	MY2+ (kNm)	MY2- (kNm)
Level 1	0.1871	0.1559	8.460E-03	0.0033	0.3991	0.3991	324	-324	324	-324
Level 2-4 Exterior	0.1497	0.1247	6.768E-03	0.0023	0.3991	0.3991	289	-289	289	-289
Level 2-4 Interior	0.1497	0.1247	6.768E-03	0.0013	0.3991	0.3991	214	-214	214	-214
Level 5 Exterior	0.1497	0.1247	6.768E-03	0.0019	0.3991	0.3991	161	-161	161	-161
Level 5 Interior	0.1497	0.1247	6.768E-03	0.0022	0.3991	0.3991	160	-160	160	-160
Level 6-R Exterior	0.1497	0.1247	6.768E-03	0.0017	0.3991	0.3991	158	-158	158	-158
Level 6-R Interior	0.1497	0.1247	6.768E-03	0.0024	0.3991	0.3991	157	-157	157	-157

Where:

A	=	Beam cross sectional area
AS	=	Effective shear area, assumed as $AS = \frac{5}{6}A$
I	=	Section moment of inertia
RF	=	Bi-linear factor (flexure), $RF = \frac{1}{k_0}(\frac{M_u - M_y}{\varphi_u - \varphi_y})$
H1	=	Plastic hinge length at end 1, $L_{pb} = 0.08L + 0.022 f_y d_b$
H2	=	Plastic hinge length at end 2
MY1+	=	Positive yield moment (end 1)
MY1-	=	Negative yield moment (end 1)
MY2+	=	Positive yield moment (end 2)
MY2-	=	Negative yield moment (end 2)

The Modified Takeda Hysteretic Rule (IHYST=4) was adopted in modelling beam behaviour under cyclic loading, illustrated as Figure 8- 13, and Table 8- 5 summarises the values of hysteretic parameters applied during modelling.

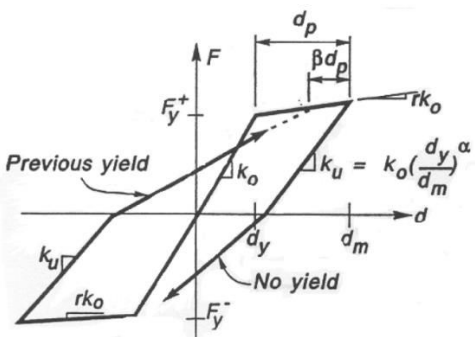


Figure 8- 13: Modified Takeda hysteretic rule (IHYST=4)

Table 8- 5: Modified Takeda hysteresis parameters applier during modelling

Parameters	α	β	NF	KKK
Value	0.3	0.6	1	2

Where:

α = Unloading stiffness
β = Reloading stiffness
NF = Reloading stiffness power factor
KKK = 1 unloading as in DRAIN-2D
2 unloading as by Emori and Schnobrich

For column elements, the interaction of moment and axial load was properly specified during modelling. Table 8- 6 provides a summary of the calculated column moment-axial load interaction characteristics.

Table 8- 6: Summary of column characteristics

Columns	**A (m)**	**AS (m^2)**	**I (m^4)**	**H1 (m)**	**H2 (m)**	**PYC (kN)**	**PB (kN)**	**MB (kNm)**	**M1B (kNm)**	**M2B (kNm)**	**M0 (kNm)**	**PYT (kN)**
Level G-4 Exterior	0.21	0.17	3.64E-03	0.27	0.27	-6904	-4621	300	480	518	300	2038
Level 5-R Exterior Level 6-R Interior	0.21	0.17	3.64E-03	0.27	0.27	-5664	-4319	130	325	357	130	679
Level G-4 Interior	0.21	0.17	3.64E-03	0.27	0.27	-6904	-4873	246	418	455	246	2038
Level 5 Interior	0.21	0.17	3.64E-03	0.27	0.27	-6284	-4688	178	356	391	178	1359

Fukada Degrading Tri-linear Hysteretic Rule (IHYST=14) was adopted in modelling column behaviour under cyclic loading, illustrated as Figure 8- 14, and Table 8- 7 summarises the values of hysteretic parameters applied during modelling.

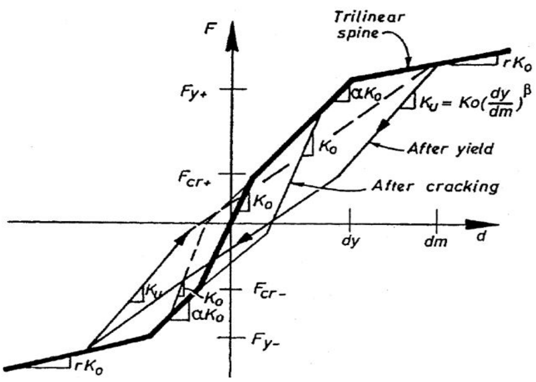


Figure 8- 14: Fukada degrading tri-linear hysteresis

Table 8- 7: Fukada hysteresis parameters applier during modelling

Parameters	α	β	FCR(i)+	FCR(i)-
Level G-4 Exterior	0.6746	0.3	28	-28
Level 5-R Exterior & Level 6-R Interior	0.2958	0.3	28	-28
Level G-4 Interior	0.5391	0.3	28	-28
Level 5 Interior	0.3937	0.3	32	-32

Where:

α = Bi-linear factor (cracking tor yield), $\alpha = \frac{1}{k_0}(\frac{M_y - M_c}{\varphi_y - \varphi_c})$
β = Unloading stiffness factor (see Takeda parameter α)
FCR(i)+ = Cracking moment or force at i
FCR(i)- = Cracking moment or force at i

8.3.3. Rigid Link Model

To connect the beam and column elements to the joint springs, rigid elements were assumed. For these rigid links, the relationship $I_{rigid} = 5I_{gross}$ was assumed.

8.3.4. Other Issues Regarding Developing Model of the Entire Structure

The following list includes some issues regarding developing the model for the entire structure, which are not discussed in the previous sections.

- It is worth noting that the upper column end was slaved to the lower column end in lateral translation and rotation. (It is worth noting that there are mistakes found in Galli's code file.)
- The linear lateral loading pattern was applied during numerical modelling, as well as during carrying out the improved simplified analytical approach, for the purpose of simplicity.
- The degradation of beam and column strength was not properly modelled. However, the analysis parameters associated with the shear strength degradation of the exterior joint proposed by Galli was applied during modelling.
- The interaction between the foundation and the super-structure was not properly modelled.
- The secondary components and non-structural components were not properly modelled.

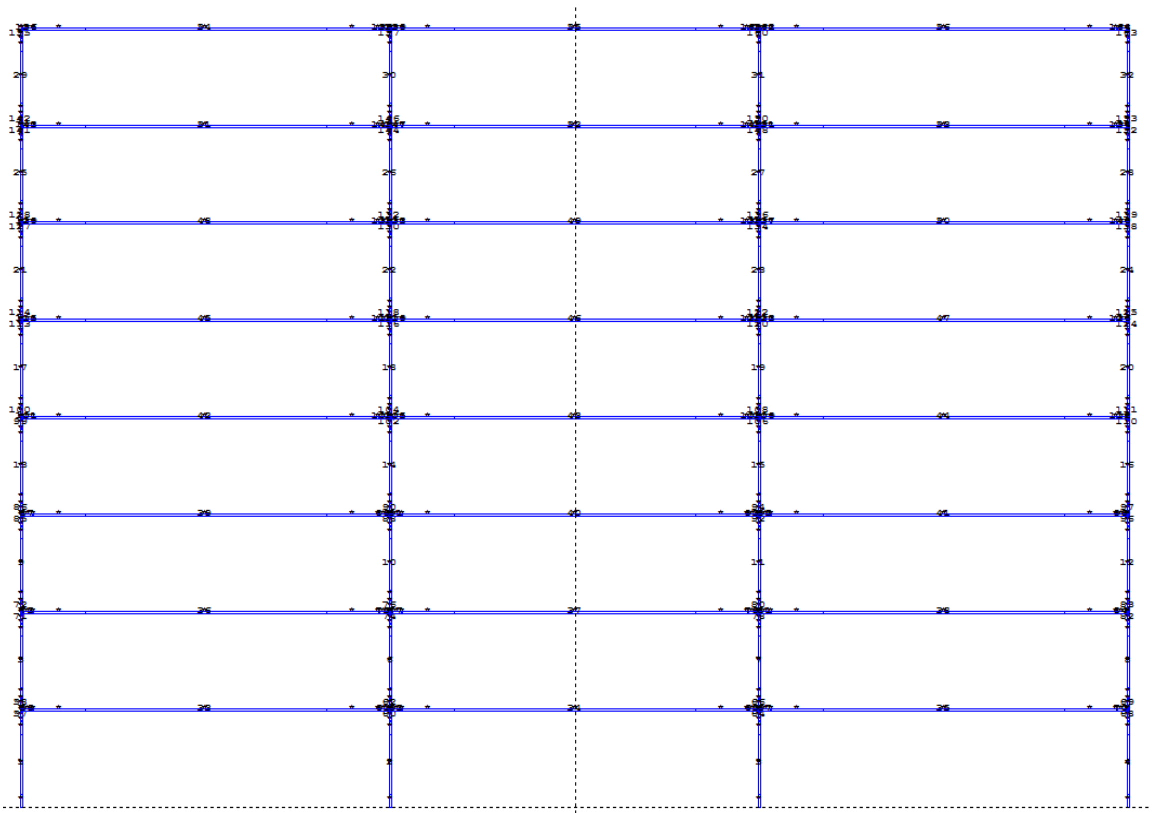


Figure 8- 15: Illustration of proposed model

8.4. Pushover and Adaptive Pushover Analysis

A nonlinear pushover analysis can be achieved by using a force excitation with a slow ramp loading function without introducing significant interior forces. According to RUAUMOKO Theory Section 7.7, two major difficulties associated with the conventional pushover analysis are:

- To assess how much to increase the load without causing the analysis to fail because the load has exceeded the capacity of the structure. During modelling, a ramp of 2kN/s was assumed.
- The pushover loading pattern applied in the normal pushover analysis may be inappropriate, especially when a structure is in its post-yield state. In Table, alternative loading patterns are shown.

Table 8- 8: Alternative pushover loading patterns (RUAUMOKO Theory, Section 7.7)

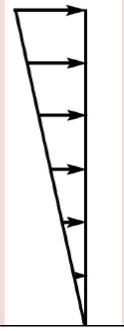
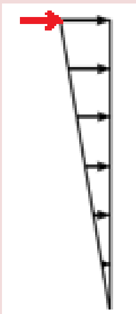
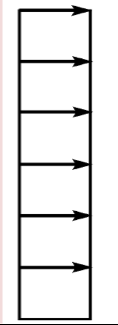
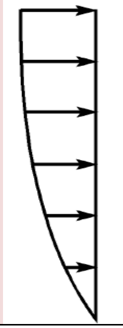
Linear defined in NZS 4203: 1992	Profile defined in NZS 1170:5	Uniform	Outward Parabolic	Inward Parabolic
				

Figure 8- 16 shows the pushover curves computed under different loading patterns for a prototype 6-storey frame analysed as an example shown in RUAUMOKO Theory, Section 7.7. Since the actual lateral load profile should vary corresponding to the displaced shape of the structure as discussed in Section 4.4, neither of the illustrated pattern in Table 8- 8 can provide a good approximation of the actual load profile. As discussed in Section 4.4, NZSEE 2006, EN 1998-3 and NTC 2008 suggest that at least two patterns should be applied in the analysis, with at least one from modal-type of patterns and at least one from uniform-type. However, ASCE 41-13 recommends that a single pattern that is based on the first mode shape should be applied. During modelling Frame 1 of Building No.21, a linear loading pattern was used, for the reason that the same linear loading pattern was applied in the improved "by-hand" pushover analysis and consistency were kept in carrying out both analyses. Therefore, it was expected that uncertainties were introduced to the modelling results. More discussions regarding the results are shown in Section 8.5.

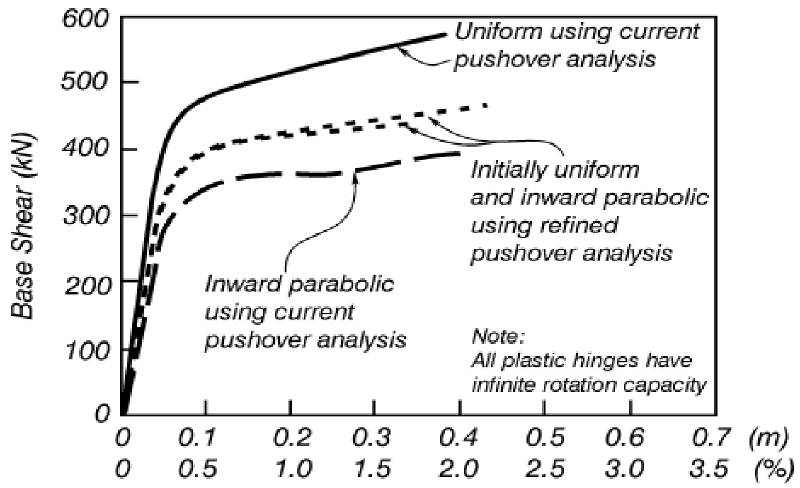


Figure 8- 16: Pushover curves computed with different load patterns for a prototype 6-storey frame (RUAUMOKO Theory, Section 7.7)

Adaptive pushover analysis, in which the input load pattern is adapted as the structure deforms, can automatically terminate when the capacity of the structure is reached, and was also applied to model Frame 1 of Building No.21. During modelling, the linear loading pattern was chosen as the initial pattern in the analysis, and the loading patterns afterwards reflected the deformation pattern of the structure, which were independent of the initial pattern. It is worth noting that the choice of the initial load pattern can influence the number of steps required to reach the structure's capacity.

Figure 8- 17 shows the pushover curves computed for the prototype 6-storey frame in RUAUMOKO Theory, by an adaptive pushover analysis and the conventional pushover analysis. As shown in Figure 8- 17, it can be deduced that the adaptive pushover analysis can yield results that are more consistent with the characteristics of the structure under consideration.

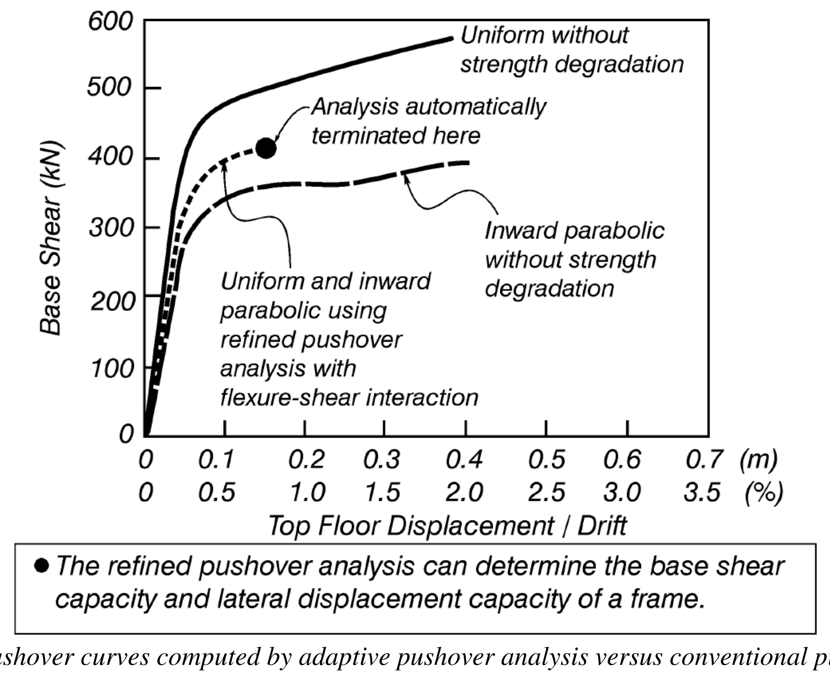


Figure 8- 17: Pushover curves computed by adaptive pushover analysis versus conventional pushover analysis for the prototype 6-storey frame (RUAUMOKO Theory, Section 7.7)

8.5. Expected Outputs and Validation of Deficiencies of the Current Approach and Proposed Improvements

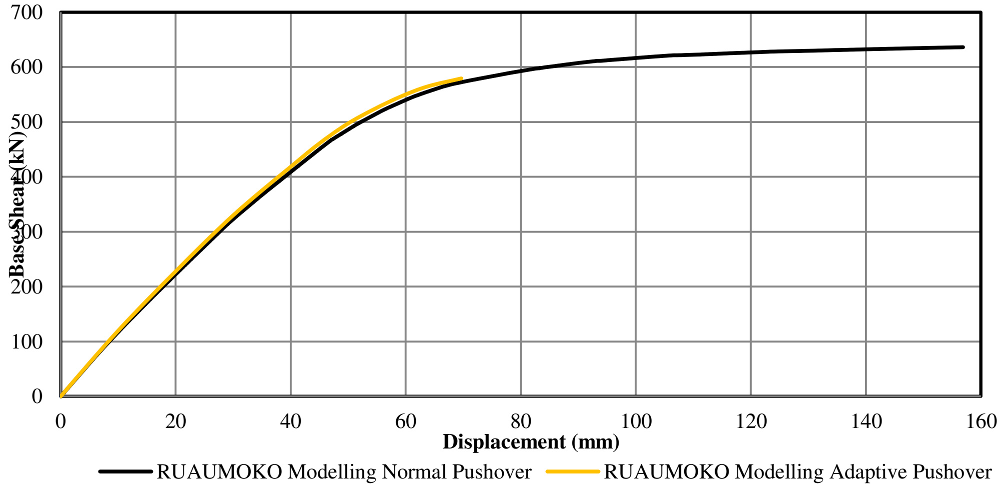


Figure 8- 18: Pushover curves computed from the conventional pushover analysis and the adaptive pushover analysis with RUAUMOKO2D for the case study structure Frame 1 of Building No.21

In Figure 8- 18, the pushover curves computed for Frame 1 of Building No.21 by a conventional pushover analysis (with the assumption of linear loading pattern) and an adaptive pushover analysis are presented. The conventional analysis yielded a lateral load capacity of 636kN, while the adaptive analysis gave 579kN, which is of approximately 15% difference. Significant difference was found in the ultimate displacement capacity. The conventional analysis yielded 157mm, while the adaptive analysis gave about 70mm.

As discussed in Section 7.3.5, the current simplified analytical approach yielded a lateral load capacity of 900kN ~ 1100kN (the values varied due to the different procedures to estimate the effective height of the structure), and it yielded an ultimate displacement capacity corresponding to the predicted failure mechanism. For instance, for the mechanism of beam sidesway without failure of joints and columns, the ultimate displacement capacity was estimated as 568mm ~ 650mm (the values varied due to the difference procedures to estimate the effective height of the structure); for the mechanism beam sidesway with interior joint failure at lower levels, the displacement capacity was approximated as 310mm; and for the mechanism with column shear failure at bottom level, the displacement capacity was assessed to be 155mm. By comparing the results from the current simplified analytical approach to those from the numerical modelling, as the pushover curves shown in Figure 8- 19, it can be inferred that the current simplified analytical approach tends to miss the correct failure mechanism, leading to considerable overestimation of the lateral load capacity and the displacement capacity of the structure. Hence, the limitations and deficiencies of the method discussed in Section 4.6.3 and Section 5.6 (Table 5- 10) are validated; in other words, the comparison between the pushover curves obtained from the numerical modelling and the current simplified analytical approach confirms the need to improve the current analytical approach.

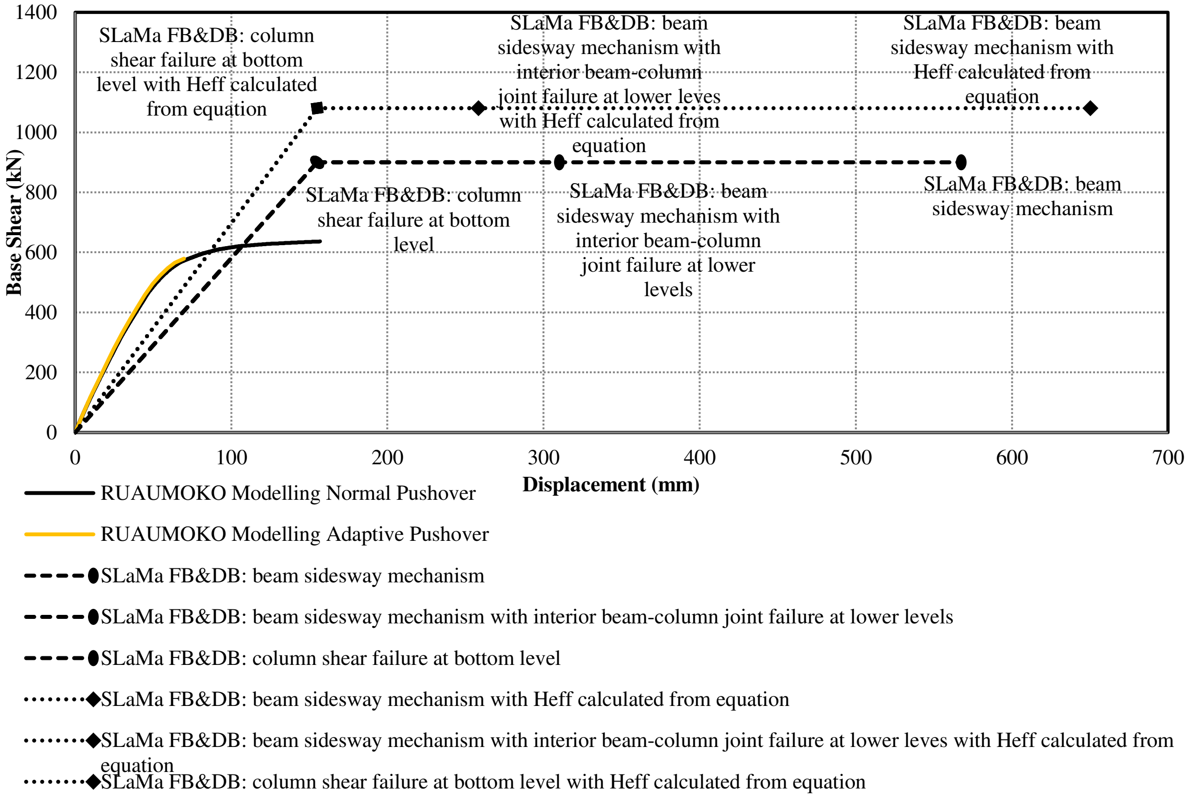


Figure 8- 19: Comparison of pushover curves computed from the current simplified analytical approach and numerical modelling for the case study structure Frame 1 of Building No.21

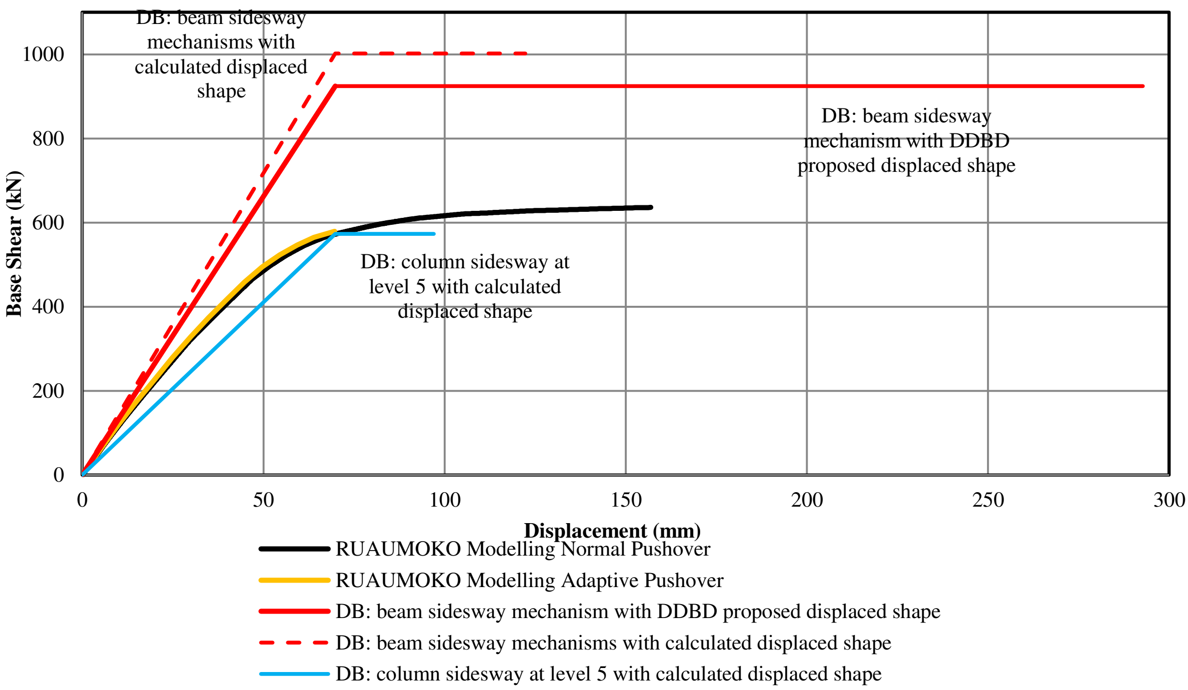


Figure 8- 20: Comparison between the upper and lower bounds computed from the improved analytical approach and the pushover curve from the numerical modelling for the case study structure Frame 1 of Building No.21

As shown in Figure 8- 20, the pushover curve computed from the numerical modelling is bounded between the lower limit and the upper limit which were analytically estimated by assuming column sidesway and beam sidesway mechanism respectively. The comparison proves the rationality of the estimated lower and upper bounds, however, it is worth recognising that the boundaries should be refined depending on the potential response of the structure. For example, if a structure is predicted to have higher possibility to undergo column sidesway mechanism, then the upper bound should be correspondingly reduced.

The sequence of mechanisms obtained from the numerical modelling is summarised in Table 8- 9. By comparing to Table 7- 28 (Section 7.3.5.3) in which the sequence predicted by the improved simplified analytical approach with Portal Frame Method is recorded, it can be confirmed that the improved approach indeed provides good approximation of the sequence of mechanisms.

Table 8- 9: Approximation of sequence of mechanisms from numerical modelling

Sequence of mechanisms from numerical modelling
Level 5 exterior beam flexural hinging
Level 5 interior beam flexural hinging
Level 2 interior beam flexural hinging
Level 3 interior beam flexural hinging
Level 6 exterior beam flexural hinging
Level 6 interior column flexural hinging
Level 4 interior beam flexural hinging
Level 2 interior column flexural hinging
Level 6 exterior column flexural hinging
Level 1 interior beam flexural hinging
Level 5 interior column flexural hinging
Analysis terminated

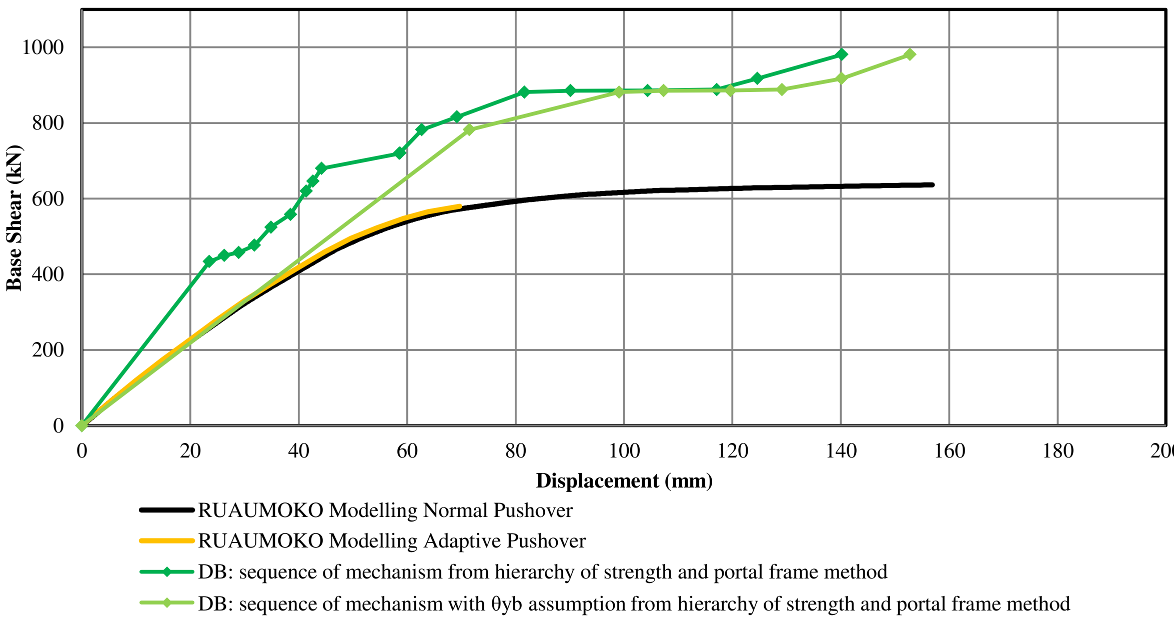


Figure 8- 21: Comparison between the pushover curves computed from the improved analytical approach with Portal Frame Method and from the numerical modelling for the case study structure Frame 1 of Building No.21

However, as shown in Figure 8- 21, the improved simplified analytical approach with Portal Frame Method yielded higher lateral load capacity (i.e. approximately 840kN, between 810kN (level 6 interior joint shear failure) and 880kN (level 5 interior column hinging)) compared to the capacity obtained from the numerical modelling (i.e. 636kN or 579kN, values varied depending on the level of sophistication of the analysis). Such significant difference may due to the listed reasons shown as following.

- In reality, one side of exterior columns should resist G+ΨcQ+E, while the other side should resist G+ΨcQ-E. However, during carrying out the improved simplified analytical approach with Portal Frame Method, the exterior columns on the two sides were assumed to resist same level axial load, i.e. G+ΨcQ+E, for the reason that the by-hand calculation process can become very complicated considering different capacities of the exterior columns on the two sides. Therefore, this simplification of calculation led to overestimation of the lateral load capacity. In Figure 8- 22, the blue curve was computed without considering the effect of axial load on the column flexural capacity (i.e. column flexural capacity computed under "zero-loading"), and a lateral load capacity of about 640kN was estimated. This provides evidence that the estimation of the lateral load capacity can be influenced significantly with the level of accuracy of the axial load assessed on the columns.
  Also, as discussed in previous chapters, the interior columns of the same level were also assumed to have same capacity, subjected to same level of axial load G+ΨcQ, ignoring the seismic induced axial loading on these interior columns.
- During carrying out the improved simplified analytical approach, no component strength degradation was taken into consideration.
- The deformation due to shear was not able to calculated with the improved simplified analytical approach, thus, this led to an underestimation of the displacement capacity and an overestimation of the lateral load capacity.
- It is worth noting that the application of the linear loading pattern should not be a cause of the difference, as in the both analytical and numerical approach, same linear loading pattern was used. In the adaptive pushover analysis, as discussed previously, the loading pattern varied with the displaced shape.

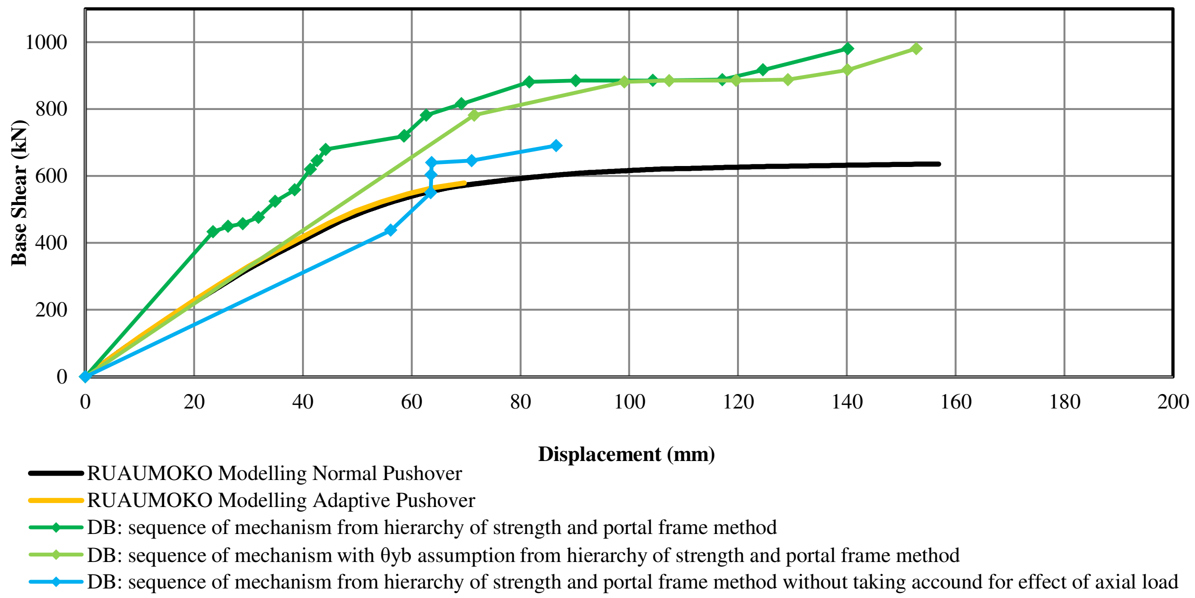


Figure 8- 22: Effect of the level of axial load resisted by exterior columns

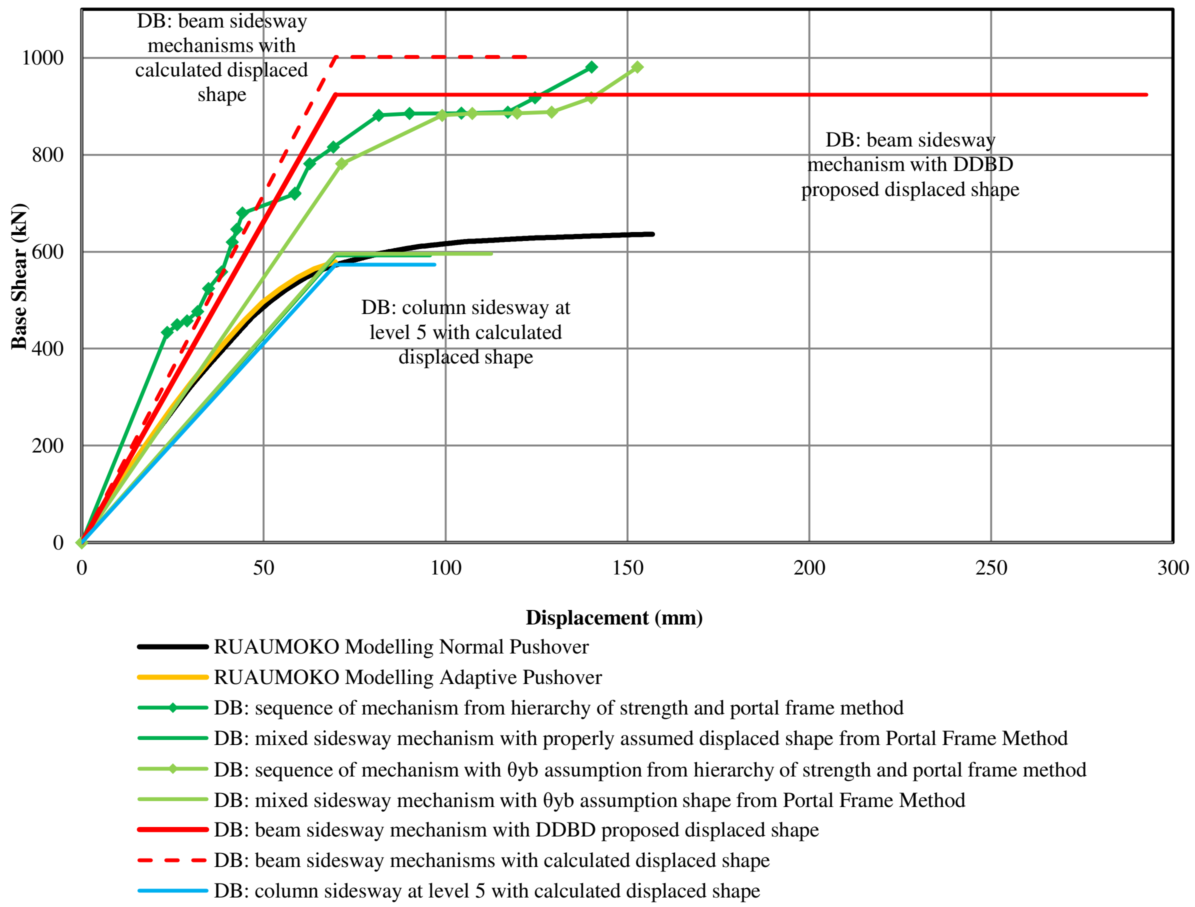


Figure 8- 23: Comparison between the pushover curves computed from the improved analytical approach with Portal Frame Method (with application of displaced shape under critical mechanism in DB approach) and from the numerical modelling for the case study structure Frame 1 of Building No.21

In Figure 8- 23, the bilinear pushover curves by applying the displaced shape corresponding to the failure mechanism predicted by Portal Frame Method in the displacement-based assessment approach is shown. By comparing to the modelling results, it can be confirmed that these bilinear pushover curves can provide good approximation of the lateral load capacity. As discussed previously, the difference in the displacement capacity estimated and the displacement given by numerical modelling is mainly due to not considering shear deformation during carrying out the improved simplified analytical approach. Hence, the rationality of the improved simplified analytical approach with Portal Frame Method (incl. Evaluation of Strength Hierarchy) is still proved.

In order to validate the improved approach that adopts Component Analysis Model and Global Structure Model, numerical modelling may needs to be conducted for all those building cases which the Models were developed from. This may involve great research effort, and should be continued in future researches.

In Table 8- 10, the results from all the simplified analytical approaches and the numerical modelling are presented.

Table 8- 10: Summary of lateral load capacity and ultimate displacement capacity

Approach	**Current SLaMa**	**Current NZSEE**	**Upper bound (DBD shape)**	**Upper bound (BS)**	**Lower bound (5CS)**	**PFM (5InCol)**	**PFM with yield assumption (5Incol)**	**Numerical modelling (conventional pushover)**	**Numerical modelling (adaptive pushover)**
Lateral Load (kN)	900	1081	924	1002	573	593	593	636	579
Yield displacement at top level (mm)	155	155	70	70	70	70	70	≈ 63	≈ 60
Ultimate displacement at top level (mm)	155, 310, 568	650	293	122	97	96	113	157	70
%NBS	23%, 45%, 83%	20%, 40%, 84%				20%	24%		

CHAPTER 9 Discussion

9.1. Overall Assessment Procedures

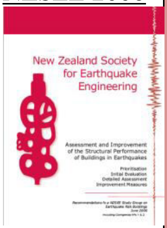
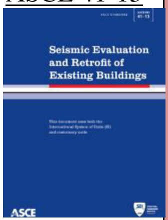
Table 9- 1: A concise flowchart of building assessment and retrofit with a detailed illustration showing assessment at component level and global level with the Portal Frame Method)

Material Level	**Section Analysis** →	**Component Level (or Subassembly Level)**	**Structural Analysis** →	**Global Level**
• Concrete • Reinforcing steel	• Moment-curvature • Force-deformation • Flexure-axial load • Flexure-shear Etc.	• Beam • Column • Joint • Wall • Masonry infill wall • Component analysis models Etc.	• Linear Static Analysis • Linear Dynamic Analysis • Simple Lateral Mechanism Analysis (current) • Improved SLaMa with the evaluation of strength hierarchy and determination of lower and upper bounds of lateral load capacity • Improved SLaMa with evaluation of strength hierarchy and Portal Frame Method • Adoption of component analysis models and global structure models • Nonlinear Static Analysis • Nonlinear Dynamic Analysis	• Frame • Frame with infills • Shear wall • Dual system • Global structure models Etc.

Table 9- 1 gives a concise flowchart of building assessment procedure. The assessment should be carried out at three levels, i.e. material level, component level (or/and subassembly level), and global level as discussed in the previous chapters. At material level, material properties (i.e. concrete and reinforcing steel for reinforced concrete structures) should be assessed following the procedures discussed in Section 3.3.3 and Section 5.2. At component level, the properties and capacities of primary components (e.g. beams, columns, joints, walls, etc.) should be determined following the procedures shown in Section 3.3.4 and Section 5.3, and the evaluation of strength hierarchy at subassembly level should be performed following the procedure shown in Section 5.4. To predict the structural response at global level based on the assessment of components, different structural analyses can be chosen from (see Section 3.3.5 and Chapter 4), among which the simplified analytical approaches are discussed in Section 4.6, 5.4 to 5.6.

Table 9- 2 provides a brief summary of codified assessment procedures, including information regarding assessment at material level, section analysis, assessment at component level (or subassembly level) and structural analysis from NZSEE 2006, ASCE 41-13, EN1998-3:2005 and NTC 2008. The detailed information together with the discussions of differences is shown in Chapter 3 and Chapter 4.

Table 9- 2: Brief summary of codified assessment procedures to determine global structural responses

Codified Assessment Procedures	Material Level	Section Analysis	Component Level (or Subassembly Level)	Structural Analysis
NZSEE 2006 	<u>Material probable strength from:</u> • Testing • Approximation • Some concrete history, information, • Some reinforcing steel history information <u>Material strain properties from:</u> • Design standards or design experiences • Mander Model or other equivalent models	<u>To calculate flexural strength:</u> • Preliminary design method (from design standards) • Hand calculation (from design standards) • Section analysis computer programs <u>To calculate shear strength:</u> specifications and formulae in design standards	**Calculation of:** <u>Beam:</u> Flexural strength Shear strength <u>Column:</u> Flexural strength Shear strength <u>Beam-Column-Joint:</u> Shear strength <u>Wall:</u> Flexural strength Shear strength	<u>NZSEE 2006 Appendix 4E:</u> • Equivalent Static Analysis • Modal Response Spectrum • Simple Lateral Mechanism Analysis (SLaMa, i.e. analytical “by-hand” pushover analysis) • Lateral Pushover Analysis • Inelastic Time History Analysis <u>NZSEE2006 Section 7 DSA:</u> • Linear Elastic Analysis • Simple Lateral Mechanism Analysis • Lateral Pushover Analysis
ASCE 41-13 	Data collection consistent with the determined knowledge level. <u>Material expected strength from:</u> (multiple of lower-bound strength and translating factor) • Default values • Full concrete history • Full reinforcing steel history • Testing <u>Material strain properties from:</u> • Design standards • Experimental evidence	Determine values of modelling parameters	Component force-deformation or normalised force-deformation ratio model with numerical acceptance criteria Modelling parameters and numerical acceptance criteria of both linear and nonlinear analysis are summarised in tables for beams, columns, joints, shear walls, slabs, foundations, infills, etc.	<u>Tier 1 Screening Procedure:</u> Simplified LSP with quick calculation <u>Tier 2 Deficiency-based Evaluation:</u> • Linear Static Procedure • Linear Dynamic Procedure <u>Tier 3 Systematic Evaluation:</u> • Linear Static Procedure • Linear Dynamic Procedure • Nonlinear Static Procedure • Nonlinear Dynamic Procedure Computer analysis programs: SAP2000, Advanced, STAAD Pro Nonlinear, PERFORM, ANSYS, etc.
EN1998-3:2005 	Material strength from: • Testing • Design standards	To calculate flexural strength, shear strength, and acceptance criteria: apply specifications and formulae in design standards	Calculation of: (acceptance criteria in terms of shear force corresponding to different limit states) <u>Beam:</u> flexural and shear strength <u>Column:</u> similar to calculation of beam <u>Beam-Column-Joint:</u> shear strength <u>Wall:</u> flexural and shear strength	<u>Knowledge Level 1:</u> • Linear Static Procedure • Linear Dynamic Procedure <u>Knowledge Level 2/3:</u> • Linear Static Procedure • Linear Dynamic Procedure • Nonlinear Static Procedure • Nonlinear Dynamic Procedure
NTC 2008	Similar to EN1998-3:2005	Similar to EN1998-3:2005	Similar to EN1998-3:2005	Similar to EN1998-3:2005

Chapter 5 presents all proposed improvements to the current New Zealand assessment guidelines, following the same order of "material level – component level – global level". The details are not repeated in this section.

It is worth noting that different levels of sophistication of assessment involve in the assessment at material level, component level or global level. The quality of the collected material information can influence the accuracy of the calculated component capacities, and hence, influence the calculation at the global level. Also, the quality of the assessed component capacities can influence the choice of analyses to be adopted to predict the global response. However, in spite of the different levels of sophistication, there is possibility that similar predictive results are still obtained. Therefore, it is important that engineers or practitioners should determine the appropriate required level of sophistication (i.e. most efficient) based on assessment or retrofit target and the available supporting resources (i.e. information, time, research efforts, money, etc.). It is also important to decide whether there is need to improve the sophistication of the assessment. For instance, if it is discovered that the application of a more sophisticate analysis can only provide similar results compared to the simplified analysis, then the simplified analysis is preferred since it is sufficient enough to meet the required level of sophistication.

9.2. Knowledge Factors and Confidence Factors

The knowledge levels specified in ASCE 41-13 and EN 1998-3: 2005 help the engineers and practitioners to determine of the appropriate level of sophistication corresponding to the available material information and other input data. These codified procedures also provide the ways to refine the available information, for instance, the procedure to conduct physical material testing, recommendation of on-site invasive investigation, and so on. However, as discussed in Section 9.1, the engineers or practitioners should be aware of the need to acquiring more information and improving the accuracy of the available information.

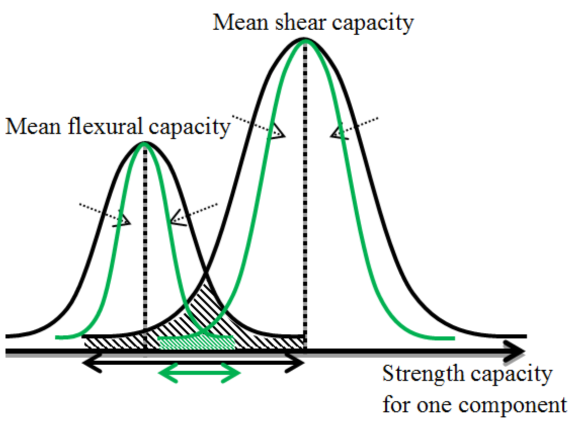


Figure 9- 1: Illustration of probable flexural and shear strengths for individual components with influence of refining input material data

As shown in Figure 9- 1, with the improvement of the accuracy of the input material information (i.e. the variation of the mean material strengths is reduced), the deviation of the mean component flexural and shear capacity are reduced (i.e. the band widths of variation rages of the mean capacities are narrowed), leading to more accurate estimation of the component capacities. The reduced over-lapped area indicates that the confidence level of calculated component capacities is increased. It is likely that the improvement does not have significant impact on the sequence of mechanisms at subassembly level (i.e. for a joint) nor at global level.

However, if the mean material strengths are altered (i.e. originally, the wrong strengths were applied in calculation, or the quality of the data was very bad), the mean component capacities will also be altered, accordingly, as shown in Figure 9- 2. It is also worth noting that the improvement of component capacity calculation may alter the mean component capacities as well. The alternation can result in either lower or higher confidence level in the outputs. For the case in which the confidence level is reduced (i.e. the overlapped area is increased), it is likely that the sequence of mechanisms at subassembly level (i.e. for a joint) can change, as shown in Figure 9- 3; and hence, it may affect the global structure response, leading to a different failure mechanism, as shown in Figure 9- 4.

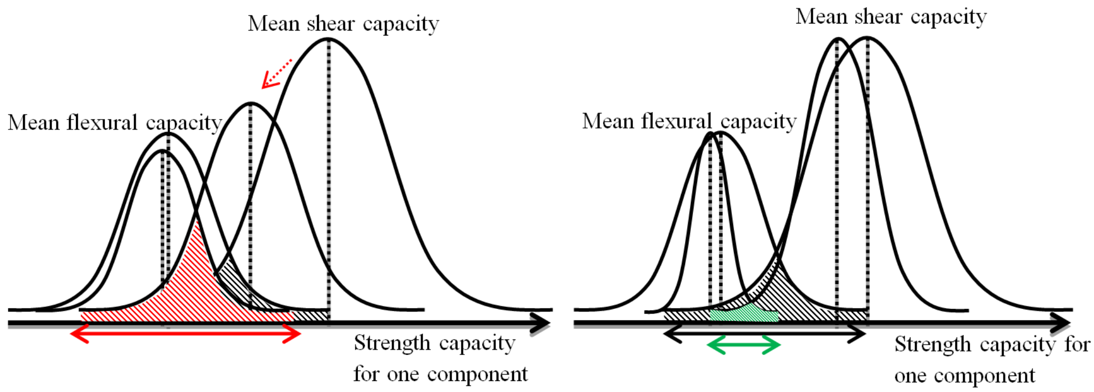


Figure 9- 2: Illustration of probable flexural and shear strengths for individual components with influences of refining input material data and improving strength calculation

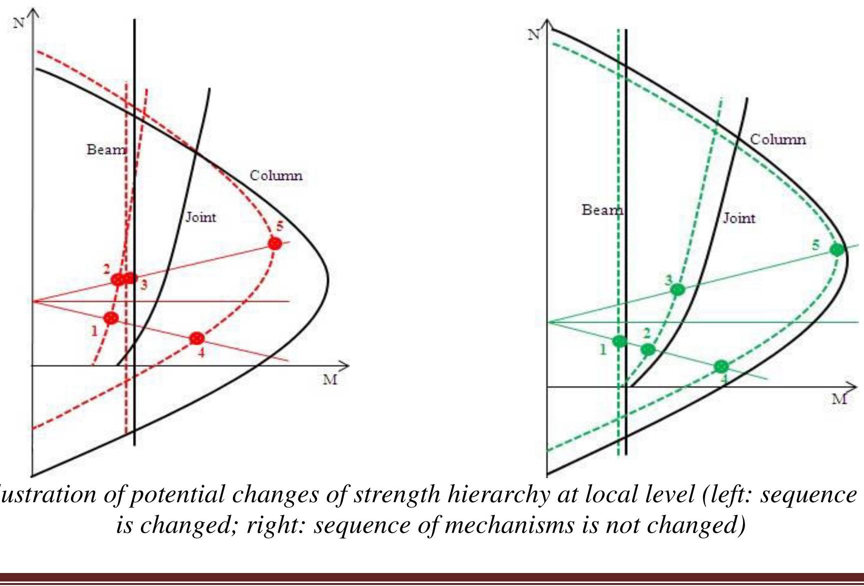


Figure 9- 3: Illustration of potential changes of strength hierarchy at local level (left: sequence of mechanisms is changed; right: sequence of mechanisms is not changed)

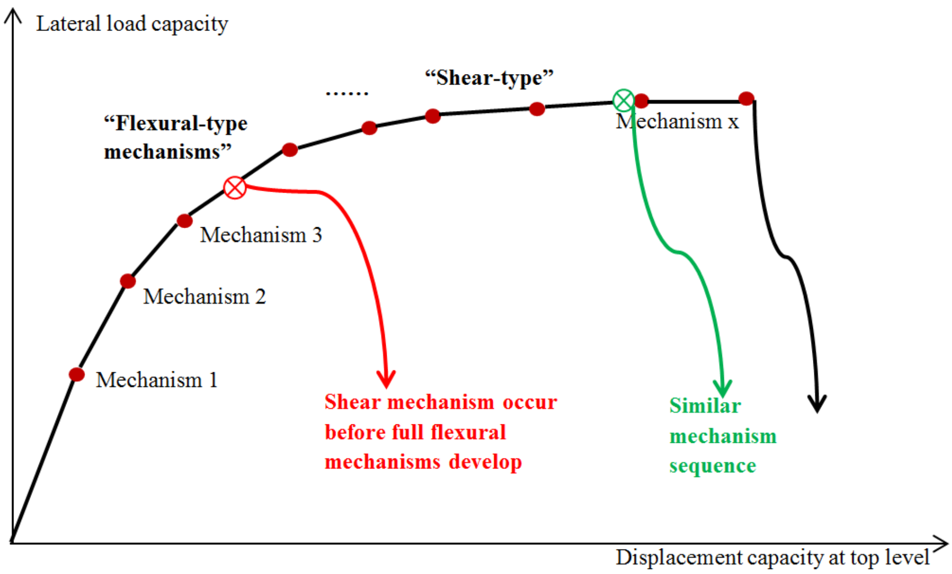


Figure 9- 4: Illustration of change of failure mechanism or sequence of mechanism at global level

Therefore, conclusion can be drawn as: if it is verified that the variations of input material data and component capacity calculation have insignificant impact on sequence of mechanisms, effort and time can be saved by only applying less sophisticated assessment without refining input data or calculation process.

In order to confirm the conclusion, sensibility test was carried out for the building case study shown in Chapter 7. The assessment results indicate that with changes of material strengths, the hierarchy of strength at subassembly level and global level was not significantly affected for the case study Building No.21 Frame 1. Further study is required to illustrate more examples where the sequence of mechanisms significantly changes due to variation of material data and component capacities.

It should be noticed that future researches are required regarding defining and application of knowledge levels and knowledge factors. SAFER Group members Alessandro De Pra and Simona Bianchi may have further investigation of this.

9.3. Demand and Capacity (FB, DB and ADRS)

In the current New Zealand assessment guidelines, acceleration spectrum and displacement spectrum is applied to determine demand. In Figure 9- 5 and Figure 9- 6, the acceleration and displacement spectrum at different ductility levels are shown.

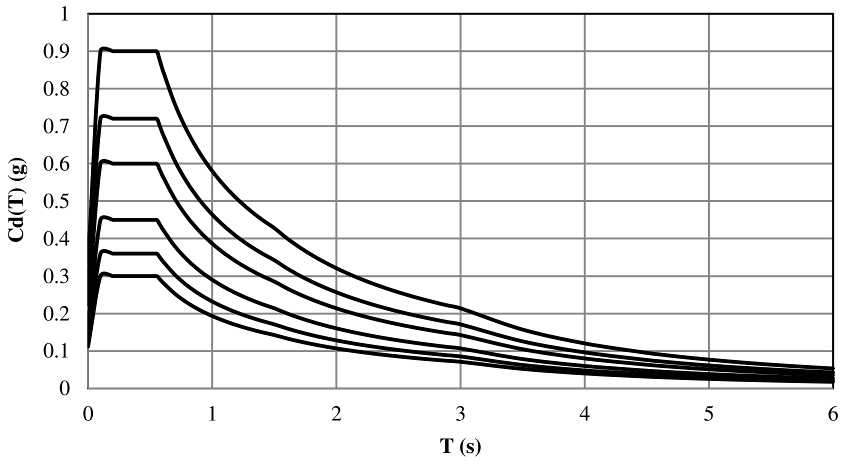


Figure 9- 5: Demand determined from acceleration spectrum

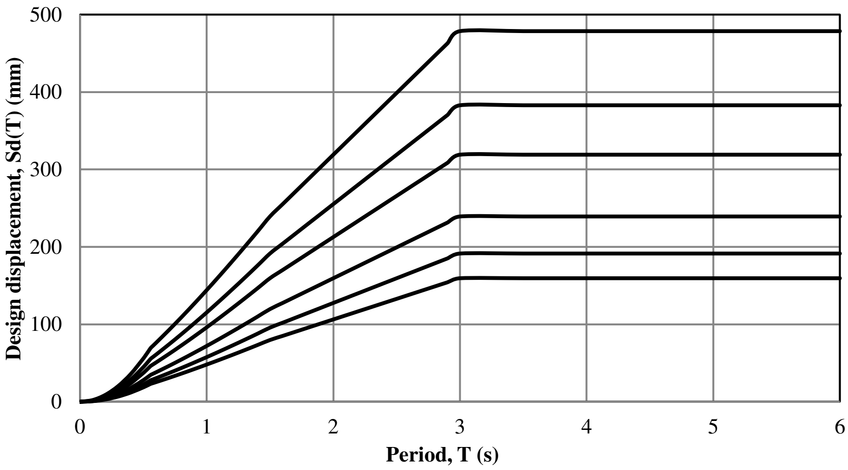


Figure 9- 6: Demand determined from displacement spectrum

It can be proposed that with the computed pushover curves (i.e. representing capacity of the structure), the acceleration-displacement format of demand (i.e. ADRS) is more straightforward to apply. In other words, the pushover curve can be plotted together with demand in the same acceleration-displacement scheme (*Displacement-Based Seismic Design of Reinforcement Concrete Buildings, FIB State of Art Report*). This can be done by dividing the computed base shear by the seismic weight. The performance point is described as the intersection of the capacity curve and the acceleration-displacement response spectrum of the effective damping ratio. %NBS can be determined by $\%\mathrm{NBS} = \frac{\mathrm{U_{sc}}}{\mathrm{U_{sd}}}$, where Usc is the ultimate displacement capacity and Usd is the displacement at performance point.

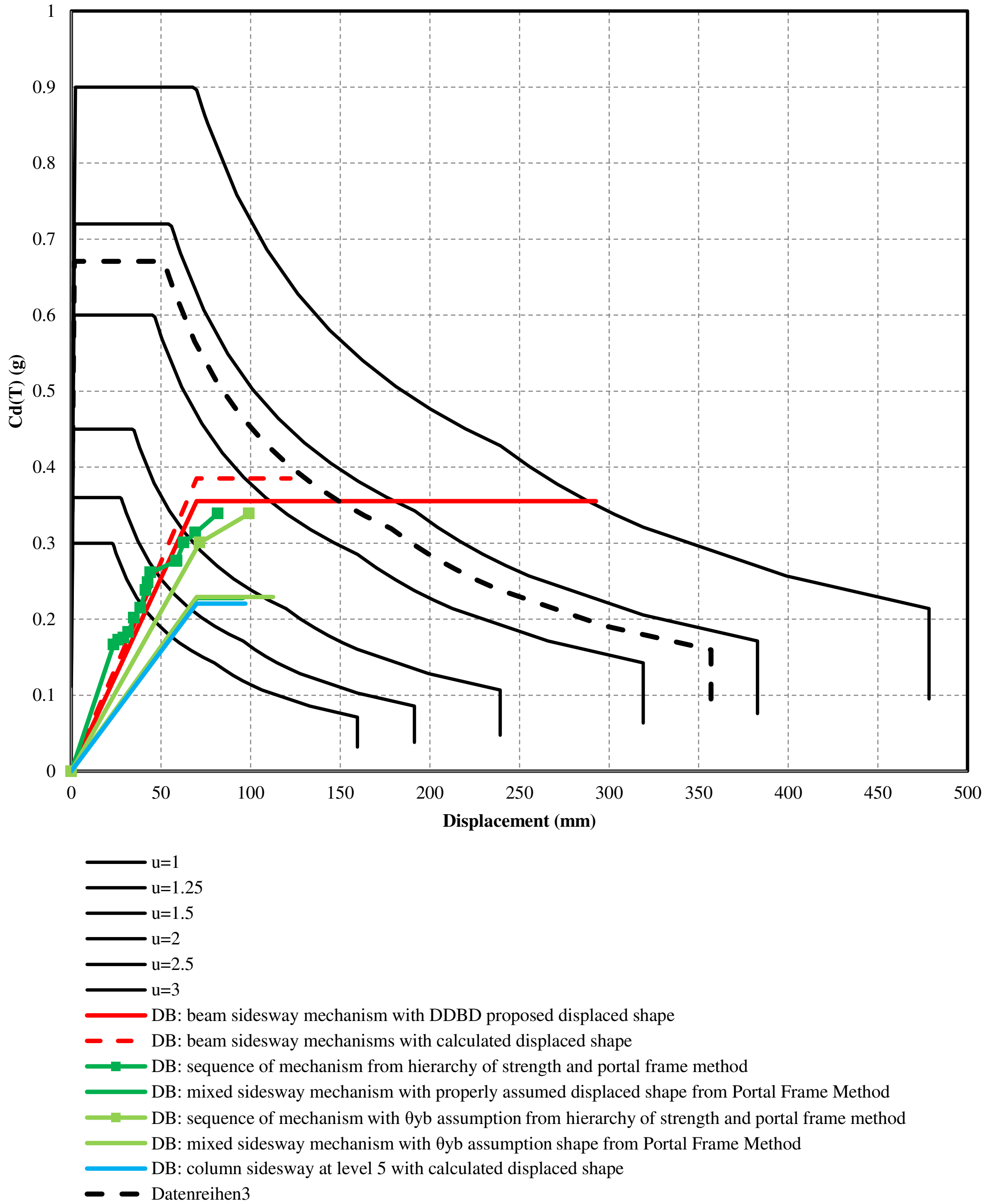


Figure 9- 7: Demand determined from capacity spectrum (ADRS format)

For mixed sidesway mechanism with properly assumed displaced shape from Portal Frame Method, it was calculated:

Approach	µsc	Cd(T)	T (s)	Sd(T) (mm)	Usc (at Heff) (mm)	Usc(at top) (mm)
Improved SLaMa with PFM – displaced shape	1.341	0.228	2.100	249.767	73.056	95.713

$$\%NBS = \frac{U_{sc}}{U_{sd}} = \frac{73.056}{249.767} = 29.250\% < 33\%$$

9.4. Application of Simplified Assessment Procedures in Practice

It has been acknowledged that if a simplified assessment procedure can provide robust estimation of actual structural response without involving complicated analysis or large amount of time and expense, such procedure is favourable in practice without doubt. However, the current SLaMa fails to compute robust outcomes, and the improved ones still need to be modified with future researches and investigations. Hence, in practice, the engineers or the practitioners prefer to conduct a numerical analysis, which usually takes more time and expense.

However, it has also been acknowledged that the application of a sophisticated analysis (e.g. numerical modelling) does not guarantee good prediction of structural response, for the reason that such analysis usually requires more understanding or knowledge of the structure, seismic characteristics, etc, as discussed in Chapter 4. One of the most critical issues is that the analysis cannot generate good results unless the components are appropriately modelled. For example, the joint failure mechanism will still be missed unless the joint regions are not properly modelled; the shear failure of column due to the interaction with infilled walls will not be predicted unless the inflled walls are correctly modelled, etc.

Therefore, it is still proposed that a simpler analytical approach should be carried out before initiating a sophisticated analysis. By performing a simpler approach, the following can be obtained:

- The quality of input data (e.g. material properties, building information, etc.) and the need of refining data or collecting more data
- The capacities of components (structural and nonstructural) and the need of refining and improving the calculation
- The possible behaviour of the structure in a seismic event
- The need of further assessment with more sophisticated analyses

If a further assessment with more sophisticated analyses is needed, the results from the further assessment should be cross-checked by the results (i.e. margins) from the simplified procedure.

CHAPTER 10 Conclusion

To conclude, this thesis provides evaluation of the capability of the simplified seismic vulnerability assessment procedures for reinforced concrete structures (mainly for frame-type of structures in this research) following the Canterbury Earthquake sequence. In each of the chapters, brief introduction, discussion and conclusive comments are included. Therefore, the details are not repeated in this final conclusion part, and brief conclusion with bullet points are shown in the following.

- The research provides a general reviews and critical comparison of the codified seismic assessment procedures. The main differences were found in:

  i. Preliminary evaluation stage;

  ii. Defining of knowledge levels and application of knowledge factors;

  iii. Assessment at material level;

  iv. Assessment at component level;

  v. Analysis approaches.

  Special focus was given regarding the simplified analytical approach adopted in the alternative assessment procedures. SLaMa was found to be the unique simplified analytical approach, from New Zealand assessment guidelines.

- Based on the understanding of the seismic assessment procedures that are available around the world, by comparing the current NZSEE 2006 assessment guidelines to the alternative procedures, the deficiencies of NZSEE 2006 were identified, and proper suggestions to improve NZSEE 2006, in this thesis, mainly SLaMa, were made.

  i. Improvement associated with the identification of the required level of sophistication in assessment, i.e. the potential adoption of knowledge levels and knowledge factors;

  i. Improvement associated with assessment at material level and component level, i.e. the determination of material and component properties and strengths, based on (1) construction documents, structural drawings, survey data, on-site investigations, physical testing, etc.; (2) the past and the current New Zealand design standards, i.e. design history; (3) the most advanced knowledge acquired from the latest researches and experimental work; (4) knowledge, along with suggestions and instructions, from the alternative assessment standards or guidelines. Detailed suggestions regarding materials (i.e. concrete and reinforcing steel) and components (i.e. mainly beams, columns, joints, and walls) were given. Discussions were made concerning the secondary structural components and nonstructural components that such components together with their interaction with the primary structural components should be properly assessed.

ii. Improvement associated with assessment at subassembly level, i.e. the evaluation of strength hierarchy.

iii. Improvement associated with assessment at global level (1) improving SLaMa with the evaluation of strength hierarchy and the determination of lower and upper bounds of lateral load capacity; (2) improving SLaMa with the determination of sequence of mechanisms by Portal Frame Method; (3) improving SLaMa with the adoption of generalised component analysis models and global structure models; (4) improving SLaMa with the application of the approach to shear wall structures. The applicability of the improvements made to SLaMa (frame) was discussed, shown in Table 5- 10.

- A brief study of building databases was provided, including introducing information of:

i. Introduction of CHCH CBD Building database and the Refined RC Building database

ii. The observed damages recorded in the building databases

iii. Building information, including (1) discussion on different knowledge levels; (2) building typology; (3) material properties and strengths; (4) component properties and strengths; (5) results from initial seismic evaluation.

- The need to improving the current SLaMa, and the capability of the different improved SLaMa procedures were evaluated carrying out the current SLaMa and the improved SLaMa procedures for case study building.

i. The deficiencies of the current SLaMa were confirmed by showing the significant overestimation of lateral load capacity and displacement capacity, with some other issues highlighted, such as the determination of effective height, the difference between the forced-based and the displacement-based procedure, etc..

ii. The improved SLaMa with the evaluation of strength hierarchy and the determination of lower and upper bounds was proved to be working well along with more sophisticated analyses.

iii. The improved SLaMa with the determination of sequence of mechanisms was proved to be robust though the large amount of hand calculation may make it difficult to apply in practice. Some other issues of Portal Frame Method were also confirmed by the outcomes, for instances, the deformation of the structure due to shear actions, assumptions made for the simplification purposes, and so on.

iv. The improved SLaMa with the adoption of generalised component models and global models was not evaluated in this research. However, it was expected this is the optimal approach that is highly applicable in practice. Future researches and investigations are required, including parametric study of building representatives selected from building database, analytical and numerical analyses of these building representatives, and so on.

v. The need to define the required sophistication depending on knowledge levels in assessment at different levels was confirmed by investigating into the influences on the assessment outcomes due to the change of material or component strengths.

- The validation of the improvements made was achieved by comparing the assessment results from the different improvement SLaMa procedures to the observed damages and the outcomes from numerical modelling.

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APPENDICES